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HYDRAULICS
LABORATORY

TECHNICAL REPORT HL-88-23

WAX LAKE OUTLET CONTROL STRUCTURE LOUISIANA

Hydraulic Model Investigation

by

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DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39181-0631

September 1988
Final Report

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Prepared for US Army Engineer District, New Orleans
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SECURITY CLASSIFICATION OF THIS PAGE

AD-A199 971

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1a. REPORT SECURITY CLASSIFICATION Unclassified	1b. RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY 2b. DECLASSIFICATION / DOWNGRADING SCHEDULE	3. DISTRIBUTION / AVAILABILITY OF REPORT Approved for public release; distribution unlimited.
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Report HL-88-23	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION USAEWES Hydraulics Laboratory 6b. OFFICE SYMBOL (If applicable) WESHS-L	7a. NAME OF MONITORING ORGANIZATION
6c. ADDRESS (City, State, and ZIP Code) PO Box 631 Vicksburg, MS 39181-0631	7b. ADDRESS (City, State, and ZIP Code)
8a. NAME OF FUNDING / SPONSORING ORGANIZATION USAED, New Orleans 8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS (City, State, and ZIP Code) PO Box 60267 New Orleans, LA 70160-0267	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. / PROJECT NO. / TASK NO. / WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification)
Wax Lake Outlet Control Structure, Louisiana; Hydraulic Model Investigation

12. PERSONAL AUTHOR(S)
Davidson, Robert A.

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Final report	FROM ____ TO ____	September 1988	50

16. SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17. COSATI CODES			18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)	
FIELD	GROUP	SUB-GROUP	Discharge coefficients	Riprap
			Hydraulic Model	Scour
			Overflow weir	Wax Lake

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

The Wax Lake Outlet Channel is an artificial channel that diverts 30 percent of the flow from the Atchafalaya River to the Gulf of Mexico. Because of degradation of the channel, its capacity to carry flow during nonflood periods has increased, resulting in lower river stages at Morgan City, Louisiana. A stone control structure was proposed to control the amount of flow passing through the Wax Lake Outlet Channel.

Tests were conducted on a 1:30-scale model that initially reproduced one-half of the rock weir notch (425 ft) and a 785-ft-long portion of the overflow rock weir at elevation 7.5 ft referred to the National Geodetic Vertical Datum. The main objectives of this model were to determine a riprap gradation that would be suitable for anticipated flows, define flow characteristics through the structure, and determine discharge coefficients for the proposed control structure.

(Continued)

20. DISTRIBUTION / AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS	21. ABSTRACT SECURITY CLASSIFICATION Unclassified
22a. NAME OF RESPONSIBLE INDIVIDUAL	22b. TELEPHONE (Include Area Code) / 22c. OFFICE SYMBOL

DD Form 1473, JUN 86 Previous editions are obsolete. SECURITY CLASSIFICATION OF THIS PAGE

Unclassified

19. ABSTRACT (Continued).

Discharge coefficients were obtained for free-flow and submerged-flow conditions for both the overflow weir and notch.

Riprap stability tests in the notch indicated that type C riprap (D_{50} minimum = 2.31 ft) would remain stable on the downstream slope if the slope was changed from a 1V on 4H to 1V on 10H.

Riprap stability tests for the overflow section indicated that a type B (D_{50} minimum = 2.05 ft) gradation would remain stable for anticipated flows.

PREFACE

The model investigation reported herein was authorized by the Headquarters, US Army Corps of Engineers, on 31 October 1984 at the request of the US Army Engineer District, New Orleans (LMN). The study was conducted by personnel of the Hydraulics Laboratory, US Army Engineer Waterways Experiment Station (WES), during the period October 1984 to December 1986. The study was conducted under the direction of Messrs F. A. Herrmann, Jr., Chief of the Hydraulics Laboratory, J. L. Grace, Jr., former Chief of the Hydraulics Structures Division, and G. A. Pickering, Chief of the Hydraulic Structures Division. Tests were conducted by Messrs. R. A. Davidson, R. G. Frazier, Jr., and M. P. Thomas under the supervision of J. F. George, Chief of the Locks and Conduits Branch. This report was prepared by Mr. Davidson.

During the course of the model investigation, Messrs. F. Chatry, C. Soileau, B. Garett, and A. Laurent of LMN visited WES to discuss model results and correlate these results with concurrent design work.

COL Dwayne G. Lee, EN, is the Commander and Director of WES. Dr. Robert W. Whalin is the Technical Director.

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CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
inches	25.4	millimetres
pounds (mass)	0.4535924	kilograms
square feet	0.092990	square metres

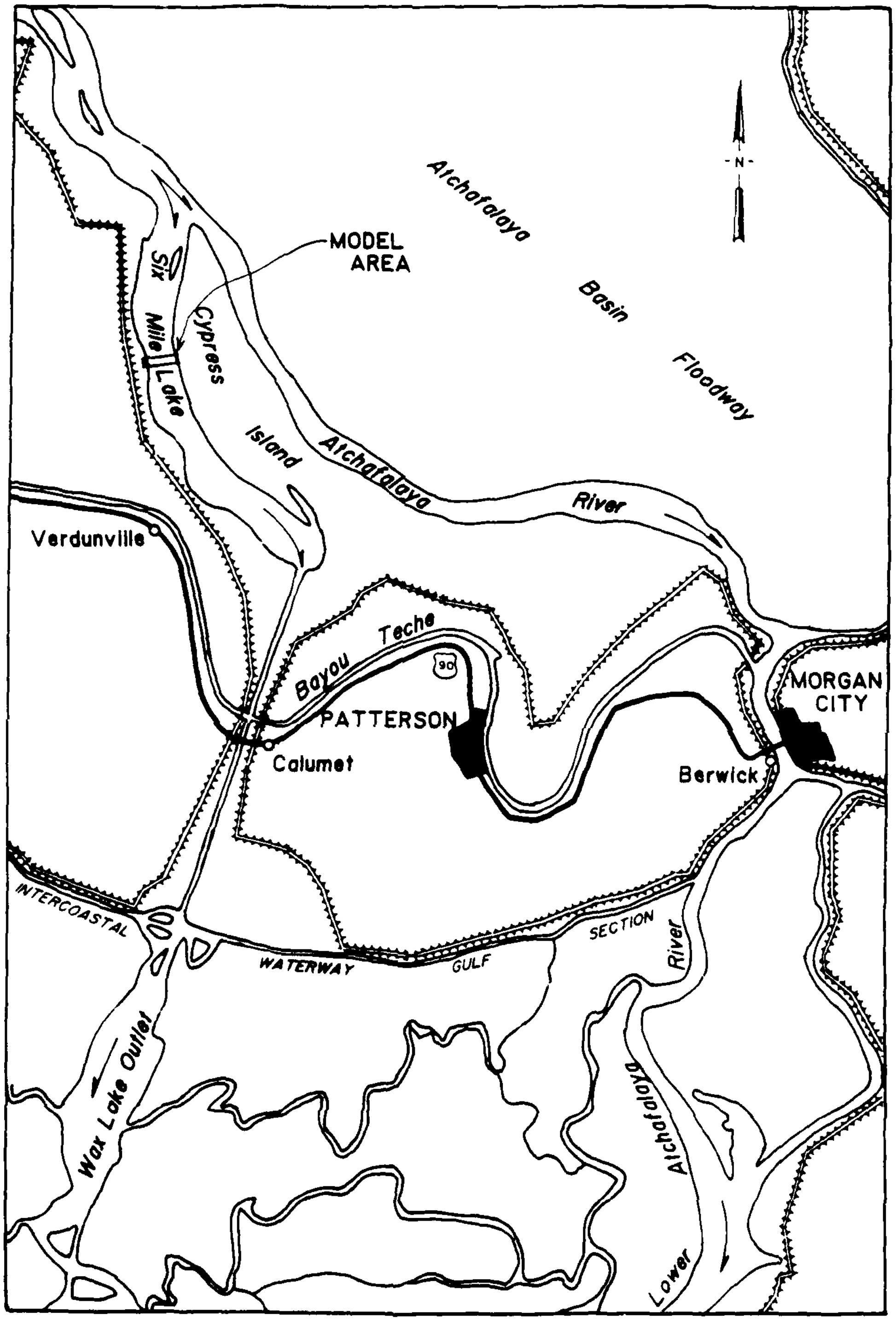

Figure 1. Location map

WAX LAKE OUTLET CONTROL STRUCTURE, LOUISIANA

Hydraulic Model Investigation

PART I: INTRODUCTION

Background

1. The Wax Lake Outlet Channel is an artificial channel that was excavated in Southern Louisiana (Figure 1) in 1942 to divert 30 percent of the flow from the Atchafalaya River to the Gulf of Mexico and reduce flood stages at Morgan City, Louisiana. Since construction, the capacity of the channel to carry flow during nonflood periods has increased due to degradation of the channel. This has resulted in lower discharges and lower river stages at Morgan City when the river discharges are below flood stage.

Purpose of the Model Study

2. A stone control structure was proposed to control the amount of flow passing through the Wax Lake Outlet Channel because of the increasing discharge through Wax Lake during nonflood flows. A model study was conducted to determine a riprap size and gradation that would be stable for anticipated flows, define flow characteristics through the structure, and determine discharge coefficients for free-flow and submerged-flow conditions for the proposed control structure.

PART II: THE MODEL

Description

3. The 1:30-scale model, shown in Figure 2, initially reproduced one-half of the rock weir notch (425 ft*) and a 785-ft-long portion of the el 7.5** overflow rock weir. The weir and the overflow section were molded in sand to the desired shape. A filter cloth was placed over the sand to serve as a filter and prevent migration of the sand through the voids of the stone. Graded riprap was then placed over the filter cloth to a required depth. Details of the original design are shown in Plate 1. Riprap gradations were provided by the New Orleans District and the prototype limits of each gradation are shown in Plates 2-4.

Model Appurtenances

4. Water used in the operation of the model was supplied by a circulating system. Discharges in the model were measured with venturi meters installed in the inflow lines and were baffled when entering the model. Water-surface elevations were measured with point gages. Velocities were measured with kent meters mounted to permit measurement of flow from any direction and at any depth. The tailwater in the lower end of the model was maintained at the desired depth by means of an adjustable tailgate.

Scale Relations

5. The accepted equations of hydraulic similitude based on the Froudian criteria were used to express mathematical relations between the dimensions and hydraulic quantities of the model and prototype. General relations for the transference of the model data to prototype equivalents are presented on page 8:

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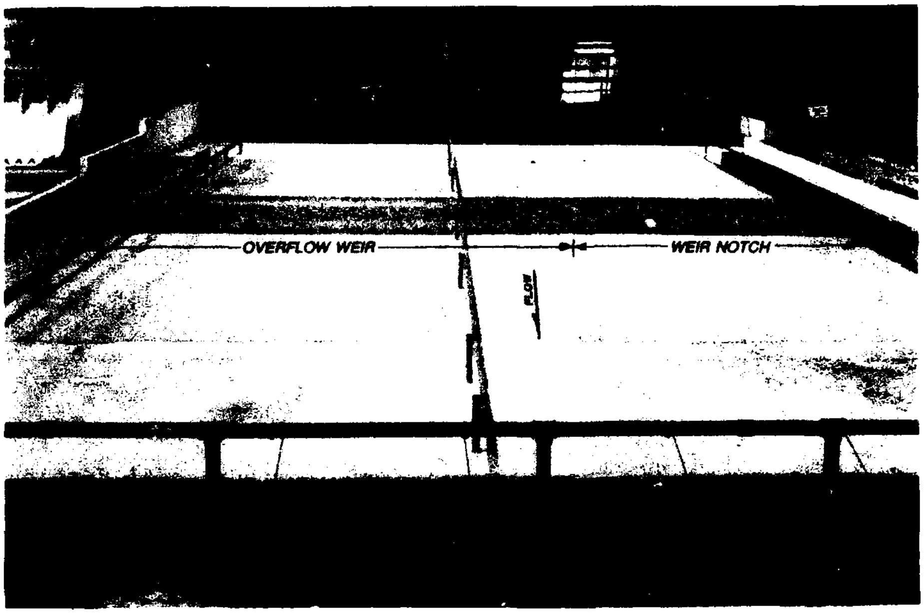
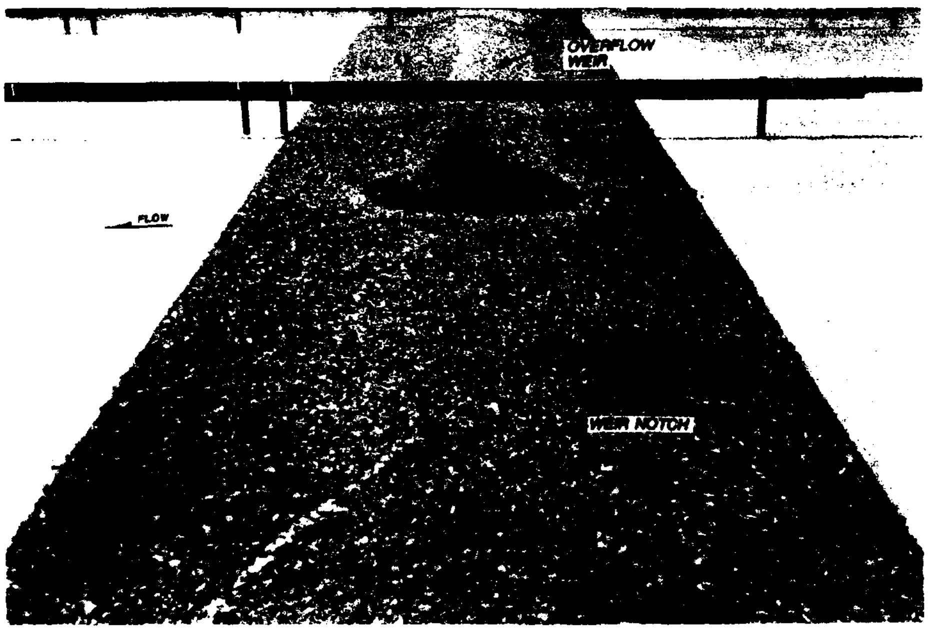
* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).



a. General view



b. Side view

Figure 2. Type 1 (original) design weir notch and overflow weir

Characteristic	Dimension*	Scale Relations Model:Prototype
Length	L_r	1:30
Area	$A_r = L_r^2$	1:900
Velocity	$V_r = L_r^{1/2}$	1:5.48
Discharge	$Q_r = L_r^{5/2}$	1:4,929.5
Time	$T_r = L_r^{1/2}$	1:5.48

* Dimensions are in terms of length.

Because of the nature of the phenomena involved, certain model data can be accepted quantitatively, while other data are reliable only in a qualitative sense. Measurements in the model of discharges, water-surface elevations, velocities, and resistance to displacement of riprap material can be transferred quantitatively from model to prototype by means of the above scale relation. Evidence of scour on the model sand bed, however, is to be considered only as qualitatively reliable since it has not yet been found possible to reproduce quantitatively in a model the resistance to erosion of fine-grained prototype bed material. Data on scour tendencies provided a basis for determination of the relative effectiveness of the different designs and indicated the areas most subject to attack.

PART III: TESTS AND RESULTS

6. Preliminary tests were conducted to obtain discharge coefficients and to observe flow through the notch and over the weir. Tests concurrently being conducted at the Waterways Experiment Station on a 1:120-scale general model of Wax Lake indicated that the notch, as originally designed, caused flow to concentrate downstream from the notch. It was determined that the notch would have to be modified in order to obtain satisfactory flow conditions immediately downstream.

Discharge Characteristics of the Overflow Weir

7. Since the elevation of the overflow weir of the structure would not change with a new notch design, the model was modified to reproduce the overflow weir (levee) at el 7.5 for the full width of the model as shown in Figure 3. Tests that were conducted to determine discharge characteristics consisted of setting a range of discharges and varying the tailwater for each from an elevation where the flow was highly submerged to an elevation where free flow occurred. A submerged-flow condition occurs when the upper pool elevation is controlled by the submergence effect of the tailwater elevation as illustrated in Photo 1. Free flow occurs when the upper pool is controlled by the discharge and is unaffected by the tailwater elevation (Photo 2). Discharge coefficients were determined for both of these flow conditions. These data are provided in Plates 5 and 6.

Free-flow coefficient

8. To determine the free-flow coefficient of the overflow weir, unit discharge q was plotted versus head above levee H (Plate 5). The slope of the line is the power function and the intercept where $H = 1$ is the free-flow coefficient. The equation for unit discharge over this part of the structure during conditions of free flow was found to be

$$q = 2.65H^{1.5} \tag{1}$$

where

q = unit discharge, cfs/ft

H = depth of headwater above levee, ft

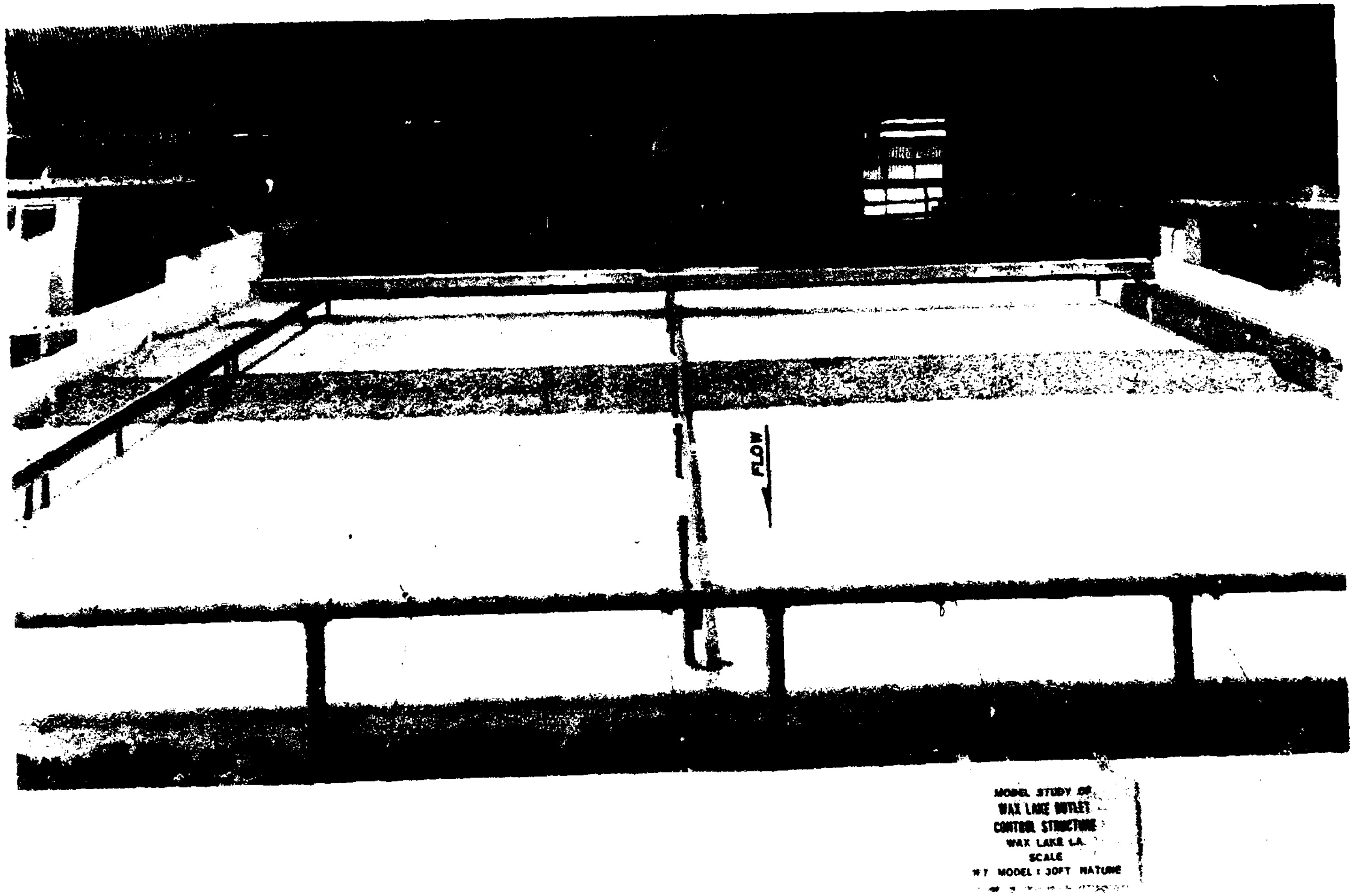


Figure 3. General view of overflow weir

Submerged-flow coefficient

9. The equation used to define submerged-flow conditions was:

$$q = C_s h \sqrt{2g\Delta H} \tag{2}$$

where

C_s = submerged-flow coefficient

h = depth of tailwater above levee, ft

g = acceleration due to gravity, ft/sec^2

$\Delta H = H - h$, ft

Since all of the terms except C_s are values obtained from the model, C_s can be solved for directly. A plot of submergence (h/H) versus submerged-flow coefficients (C_s) is provided in Plate 6. This plot indicates that tailwater begins to affect the head on the levee at h/H values of approximately 0.7 and greater, and the free-flow equation should be used to compute discharge with values less than these.

Plan A-1

Description of A-1 design

10. A new weir notch design, developed from test results from the general model, was installed in the section model (Photo 3). The length of the notch was increased from 850 to 1,125 ft and its elevation was increased from el -5.0 to el 0.0. Also the length of the el 7.5 levee was shortened. This design was designated Plan A-1 (Plate 7).

Notch riprap stability

11. Tests were conducted to determine the stability of the riprap in the notch of the Plan A-1 design by setting various discharges in the model with a high tailwater and gradually lowering the tailwater until movement of the riprap was observed. Type B riprap (Plate 3) was initially placed in the notch and was found to be unstable on the downstream slope for average tailwater conditions that can be expected at the prototype structure (Plate 8). These average headwater (HW) and tailwater (TW) curves shown in the plates were developed from the data obtained from the general model. Minimum tailwater could be considerably lower than the average curve, especially with the lower discharges. Type C riprap (Plate 4) was then placed in the notch. Stability

tests indicated that this riprap would be stable for average tailwater conditions (Plate 9). However, it probably would not be stable on the downstream slope for low tailwater conditions that could occur at the prototype structure.

12. The downstream slope of the notch in Plan A-1 was changed from a 1V on 4H slope to a 1V on 10H slope. Stability tests conducted with the type C gradation indicated that the rock would be stable for lower tailwater elevations than in previous tests (Plate 10). However, if the tailwater at the prototype structure should become more than 0.5 ft lower than the average tailwater value, the stability of the riprap is questionable.

Plan A-2

Description of A-2 design

13. The elevation of the notch was lowered from el 0.0 to el -2.0 in the Plan A-2 design based on additional test results from the general model. The notch was installed for the full width of the model (Photo 4) and tests were conducted to determine the discharge characteristics for this portion of the structure. The test procedure was the same as that used with the overflow weir in that a number of discharges were set and the tailwater depth was varied for each discharge from an elevation where the flow was highly submerged to free-flow conditions (Photo 5).

Discharge Characteristics

14. Discharge coefficients for the notch were determined for free- and submerged-flow conditions using Plan A-2 and are provided in Plates 11 and 12, respectively. From the data recorded, the equation for unit discharge during conditions of free flow was found to be

$$q = 2.75H^{1.5} \tag{3}$$

15. To determine the submerged-flow coefficients, the equation used was:

$$q = C_s\, h \sqrt{2g\Delta H} \tag{4}$$

A plot of submergence (h/H) versus the submerged-flow coefficient (C_s) (Plate 12) indicated that the depth of the tailwater begins to affect the head

on the notch at h/H values of approximately 0.7 and greater, and the free-flow equation should be used to compute discharges with values less than this.

Notch riprap stability

16. A 785-ft-long portion of the overflow weir and a 425-ft-long portion of the notch were reproduced to determine riprap requirements for average and below average tailwater conditions using Plan A-2. Riprap stability tests indicated that the type C riprap would be stable for average tailwater conditions but would be questionable for low tailwater conditions, as shown in Plate 13.

17. The downstream slope of the notch in Plan A-2 was modified from a 1V on 4H to a 1V on 10H slope. Riprap stability tests conducted with the type C riprap gradation in place indicated that the rock would be stable for tailwater conditions 3 to 4 ft below average (Plate 14). This design was stable at lower tailwater elevations than any of the designs previously tested.

Scour tests

18. The topography immediately upstream and downstream of the structure was replaced with sand in order to observe scour patterns downstream of the structure. Discharges of 30,000, 90,000, and 160,000 cfs were tested for a duration of 5.5 prototype hours each in an effort to determine if there would be an indication of a scour problem downstream of the structure using Plan A-2. With a total discharge of 30,000 cfs through Wax Lake, little or no scour was observed, but with discharges of 90,000 cfs and 160,000 cfs, a noticeable scour pattern formed downstream of the structure in the vicinity of where the notch and overflow weir connect (Photos 6 and 7). Velocities, which were obtained with the three discharges at various depths (Plates 15-22), also indicated the same type of flow pattern where the overflow weir and notch connect. Test results indicated that the riprap should be extended further downstream of the structure in this vicinity. However, the length of the riprap required was not determined because of model limits.

Overflow weir riprap stability

19. The Plan A-2 model was modified to include the el 7.5 levee for the full width of the model to conduct stability tests using types A (Plate 2) and B (Plate 3) riprap gradation. Tests indicated that the type B gradation was slightly superior to type A gradation in that it withstood a lower tailwater elevation for the same discharge. The stability curves for the two riprap gradations are shown in Plates 23 and 24.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

20. Discharge coefficients were determined for both free- and submerged-flow conditions for both the overflow weir and notch. The unit discharge over the levee and through the notch can be obtained by using the following equations for free-flow and submerged-flow conditions, respectively:

$$q = CH^{1.5} \tag{5}$$

where C is the coefficient of discharge for free flow

$$q = C_s h \sqrt{2g\Delta H} \tag{6}$$

where C_s is a function of h/H .

The submerged-flow equation should be used to calculate the amount of flow over the levee or through the notch for h/H values greater than 0.7. For h/H values less than 0.7, the free-flow equation for either the levee or the notch should be used. By using the above equations with the discharge coefficients determined from the model, the discharge over the structure can be determined for any given headwater and tailwater elevations.

21. Riprap stability tests in the notch for Plan A-1 indicated that the type C riprap gradation placed on the downstream side of the berm in the notch would be stable for average tailwater conditions, but probably would not be stable for low tailwater conditions that could occur at the prototype structure.

22. When the downstream slope of the berm in the notch for Plan A-1 changed from a 1V on 4H slope to a 1V on 10H slope, the riprap was stable for tailwater elevations 0.5 ft lower than the average, but for tailwater elevations lower than this, the stability of the riprap would be questionable.

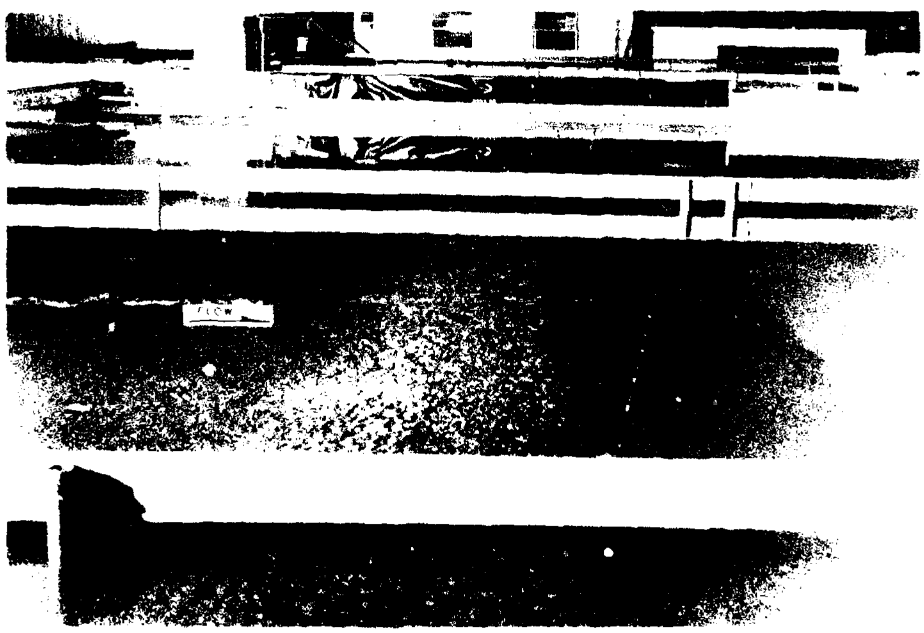
23. Riprap stability tests in the notch for Plan A-2 indicated that if the downstream slope was changed from a 1V on 4H slope to a 1V on 10H slope, the type C riprap would be stable for tailwater conditions lower than those which would occur at the structure.

24. Riprap stability tests for the levee overflow section of the structure indicated that a type B gradation would be needed to withstand conditions

that would be expected to occur at the prototype structure.

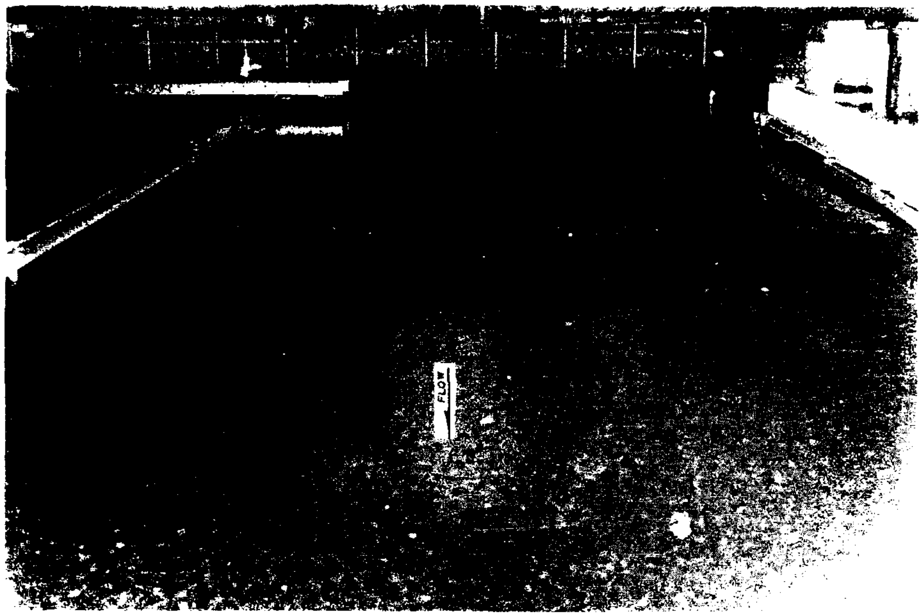
25. Excessive model scour was observed and indicated that the potential exists for a scour problem downstream of the structure in the vicinity of where the notch and the overflow weir connect. The riprap should be extended farther downstream in this area than was initially proposed.

a. General view



b. Side view

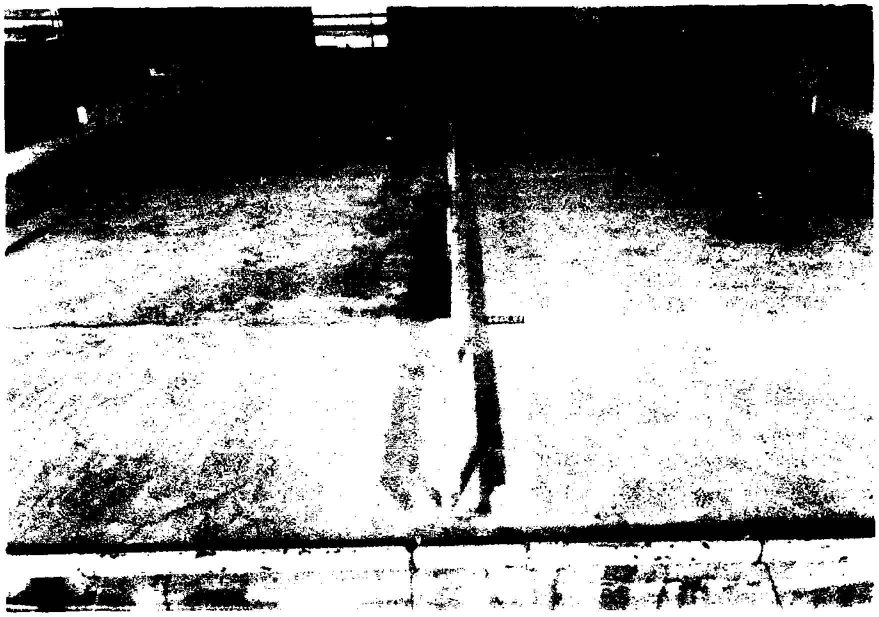
Photo 1. Submerged-flow conditions over the overflow weir (el 7.5)



a. General view

b. Side view

Photo 2. Free-flow conditions over the overflow weir (el 7.5)

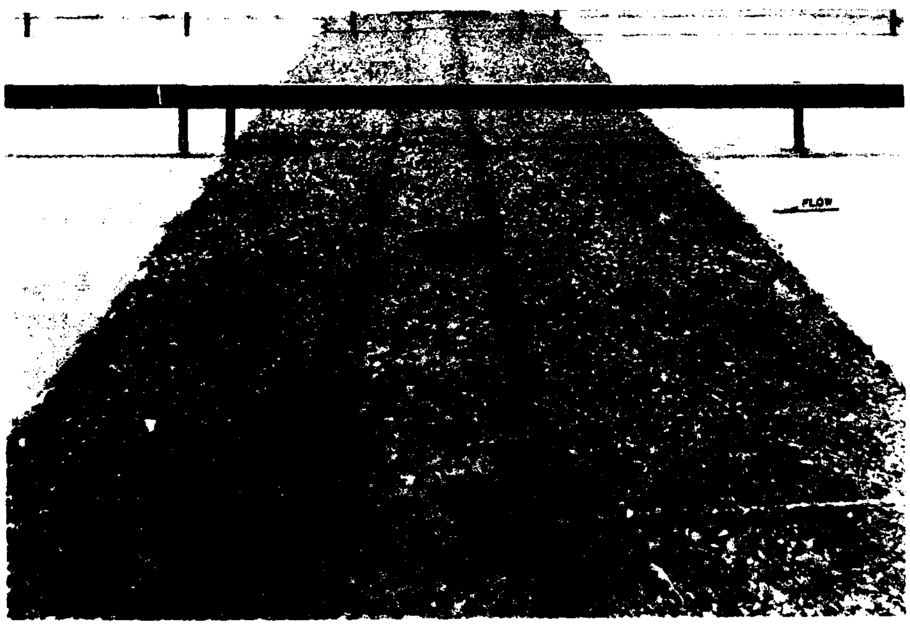


a. General view

b. Side view

Photo 3. Plan A-1 with a 1V on 4H slope on the downstream side of notch

a. General view



b. Side view

Photo 4. Plan A-2

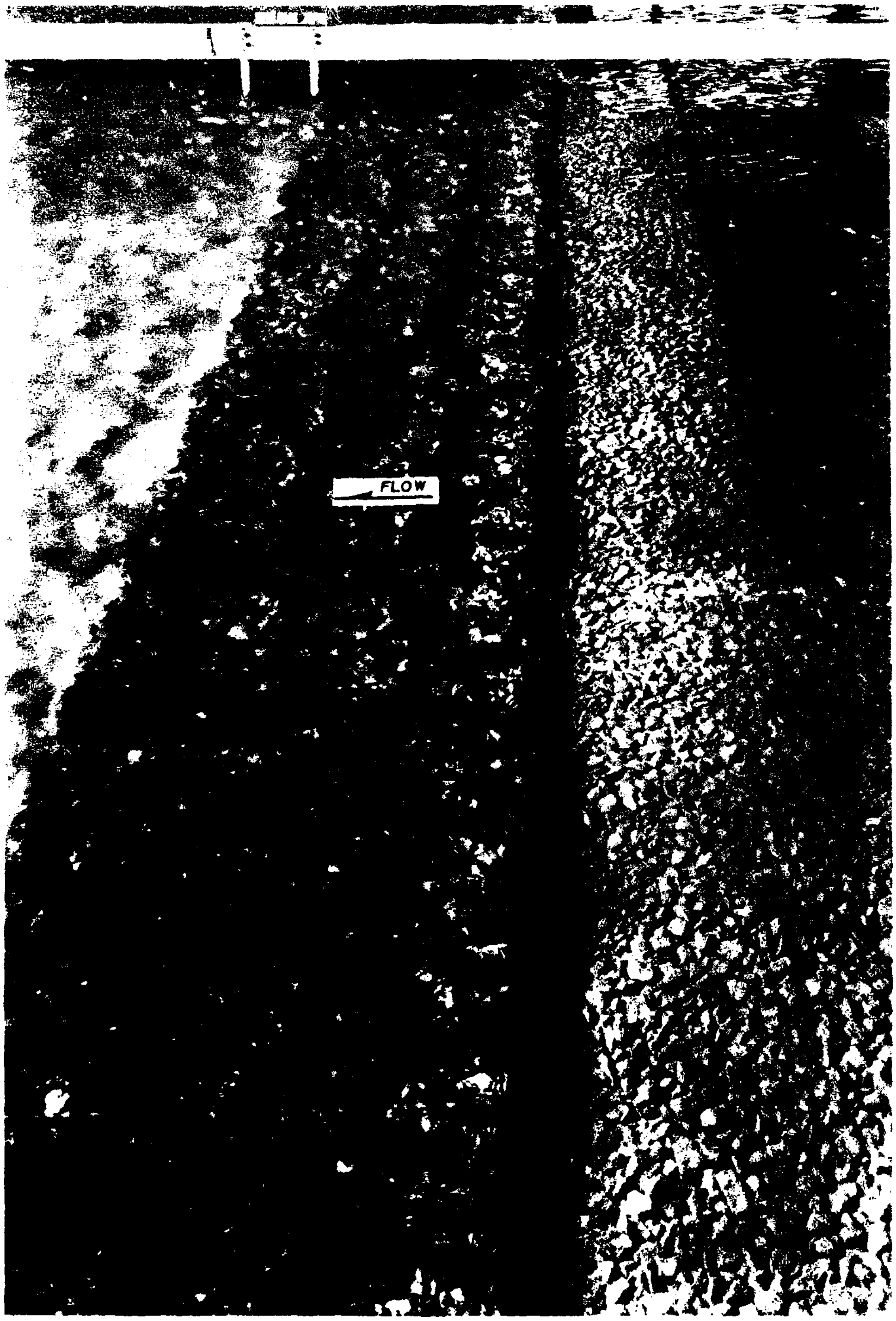
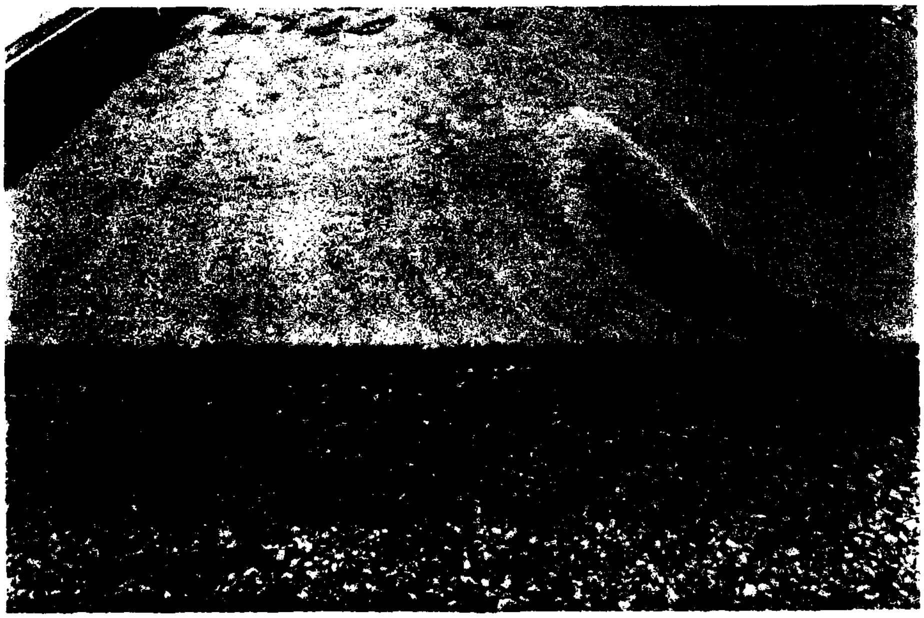


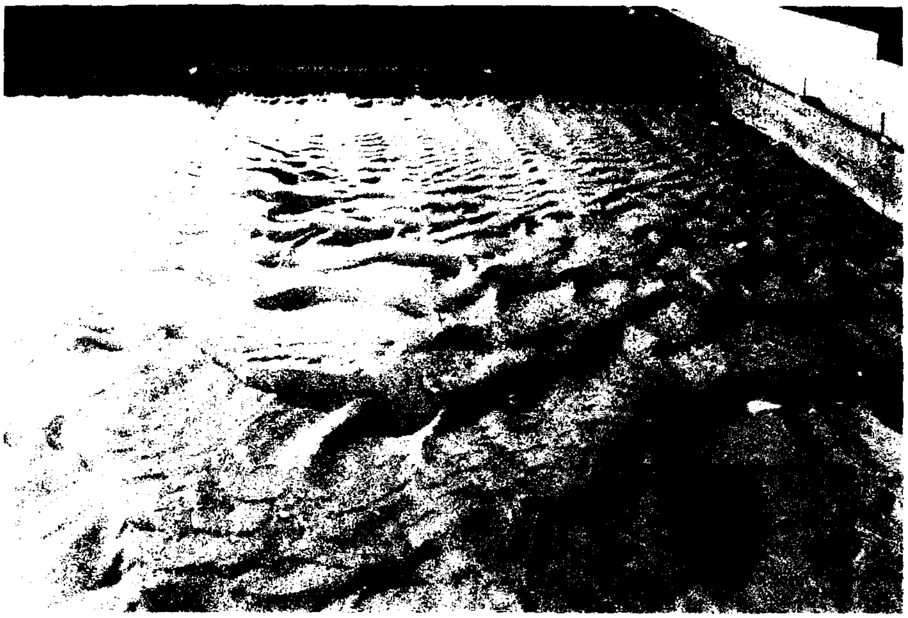
Photo 5. Free-flow conditions through the notch in Plan A-2

a. Upstream view

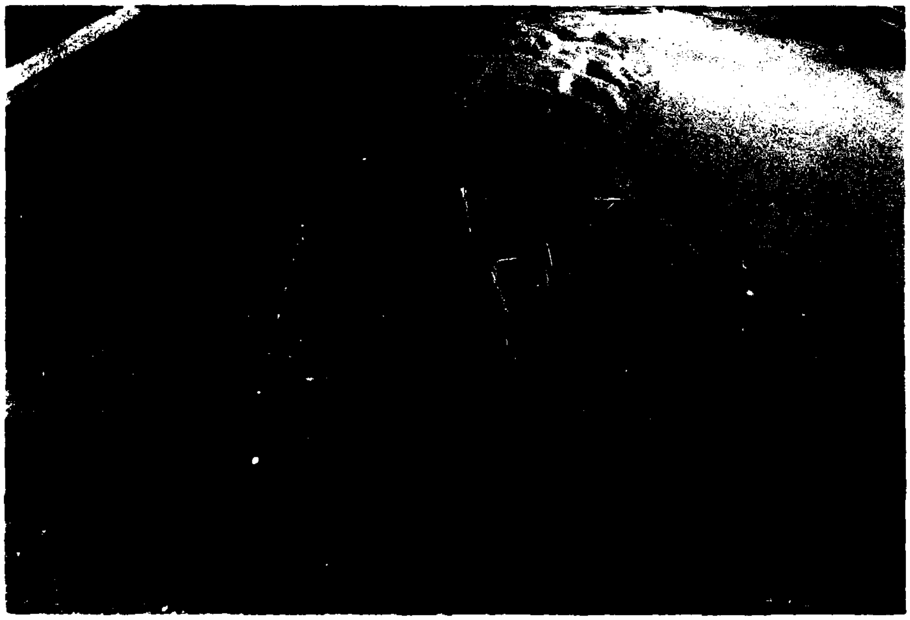


b. Downstream view

Photo 6. Scour pattern downstream of structure after a 5.5-hr (prototype) test with a discharge of 90,000 cfs

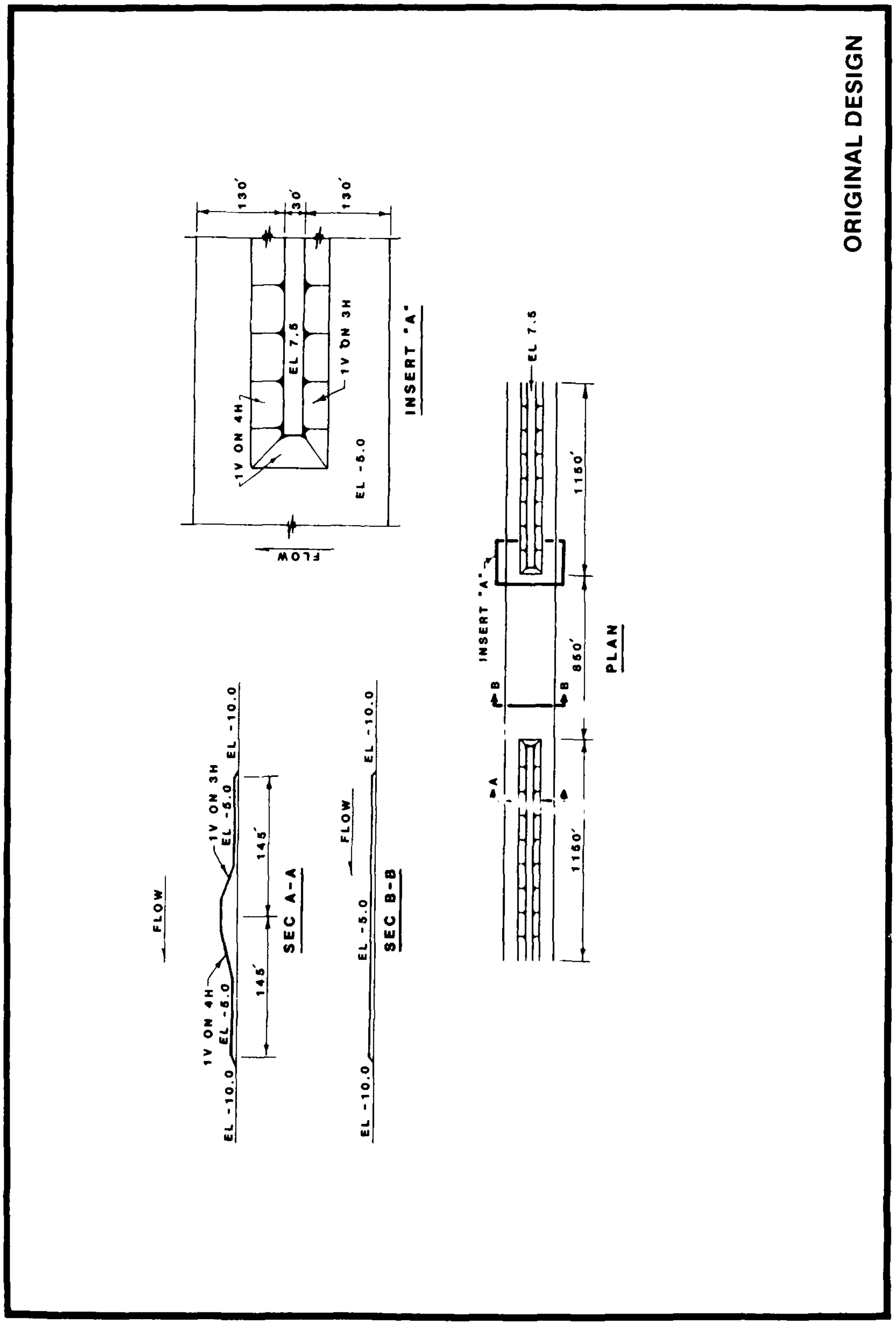


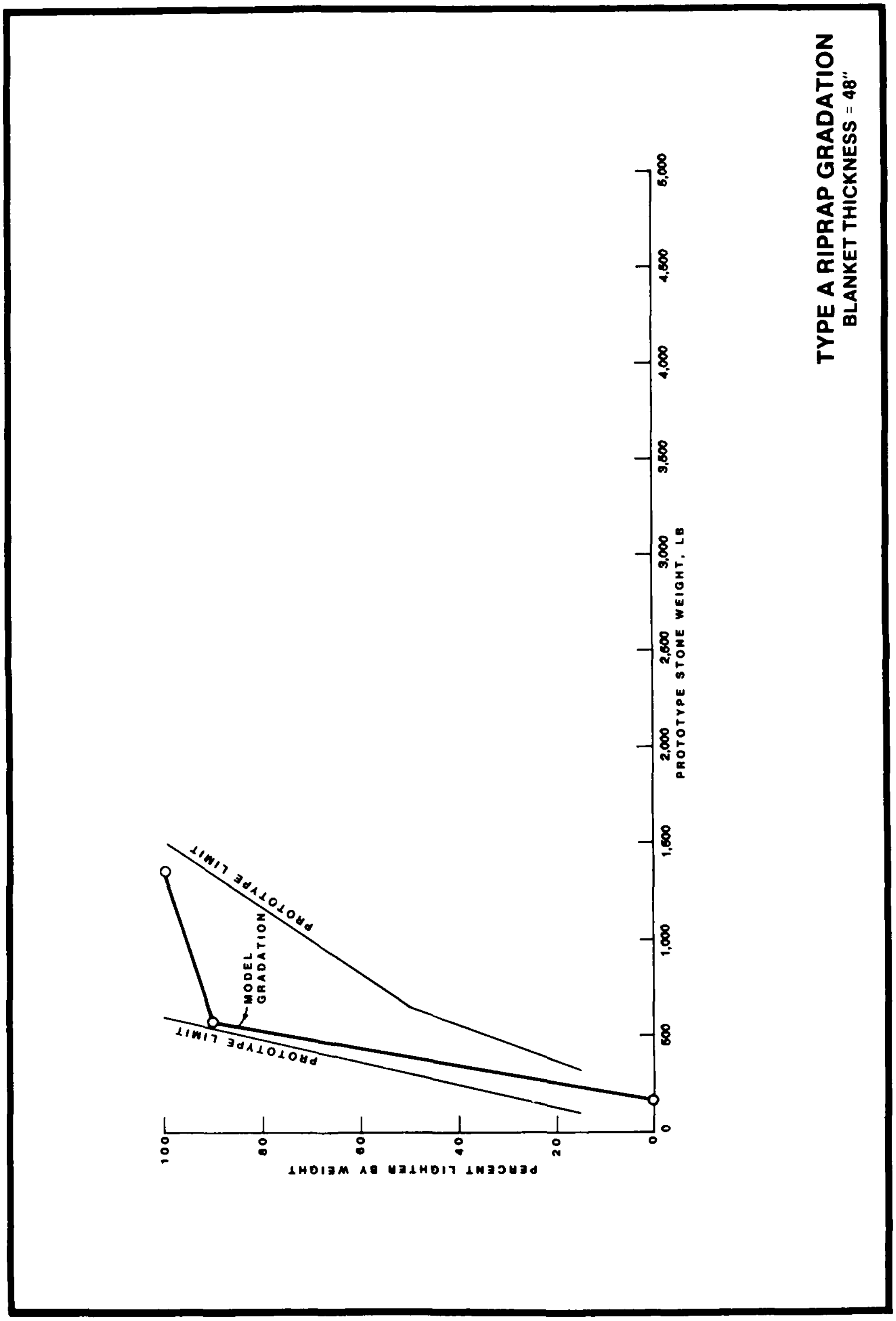
a. Upstream view

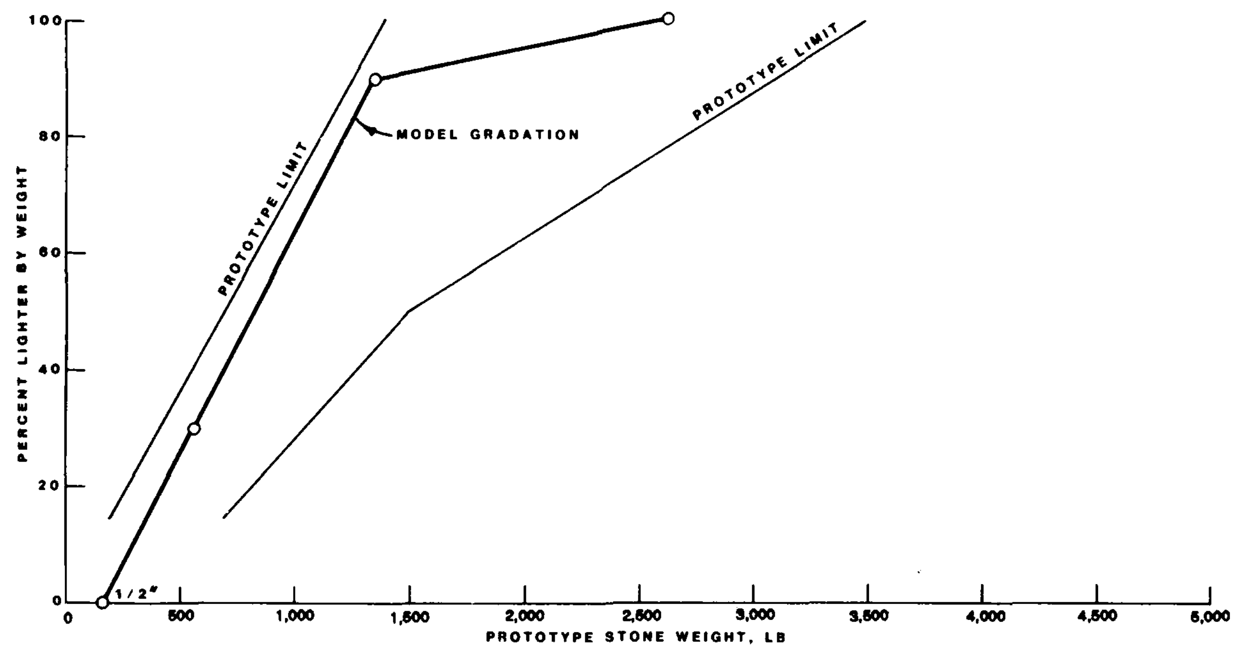


b. Downstream view

Photo 7. Scour pattern downstream of structure after a 5.5-hr (prototype) test with a discharge of 160,000 cfs


ORIGINAL DESIGN
INSERT 'A'
130'
30'
130'
EL 7.5
1V ON 4H
1V ON 3H
EL -5.0
FLOW
PLAN
INSERT 'A'
1150'
850'
1150'
B
B
A
A
EL 7.5
SEC A-A
EL -10.0
EL -5.0
1V ON 4H
1V ON 3H
145'
145'
FLOW
SEC B-B
EL -10.0
EL -5.0


TYPE A RIPRAP GRADATION
BLANKET THICKNESS = 48"
PROTOTYPE STONE WEIGHT, LB
PERCENT LIGHTER BY WEIGHT
PROTOTYPE LIMIT
PROTOTYPE LIMIT
MODEL GRADATION
0
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
4,500
5,000
0
20
40
60
80
100



TYPE B RIPRAP GRADATION
BLANKET THICKNESS = 63"

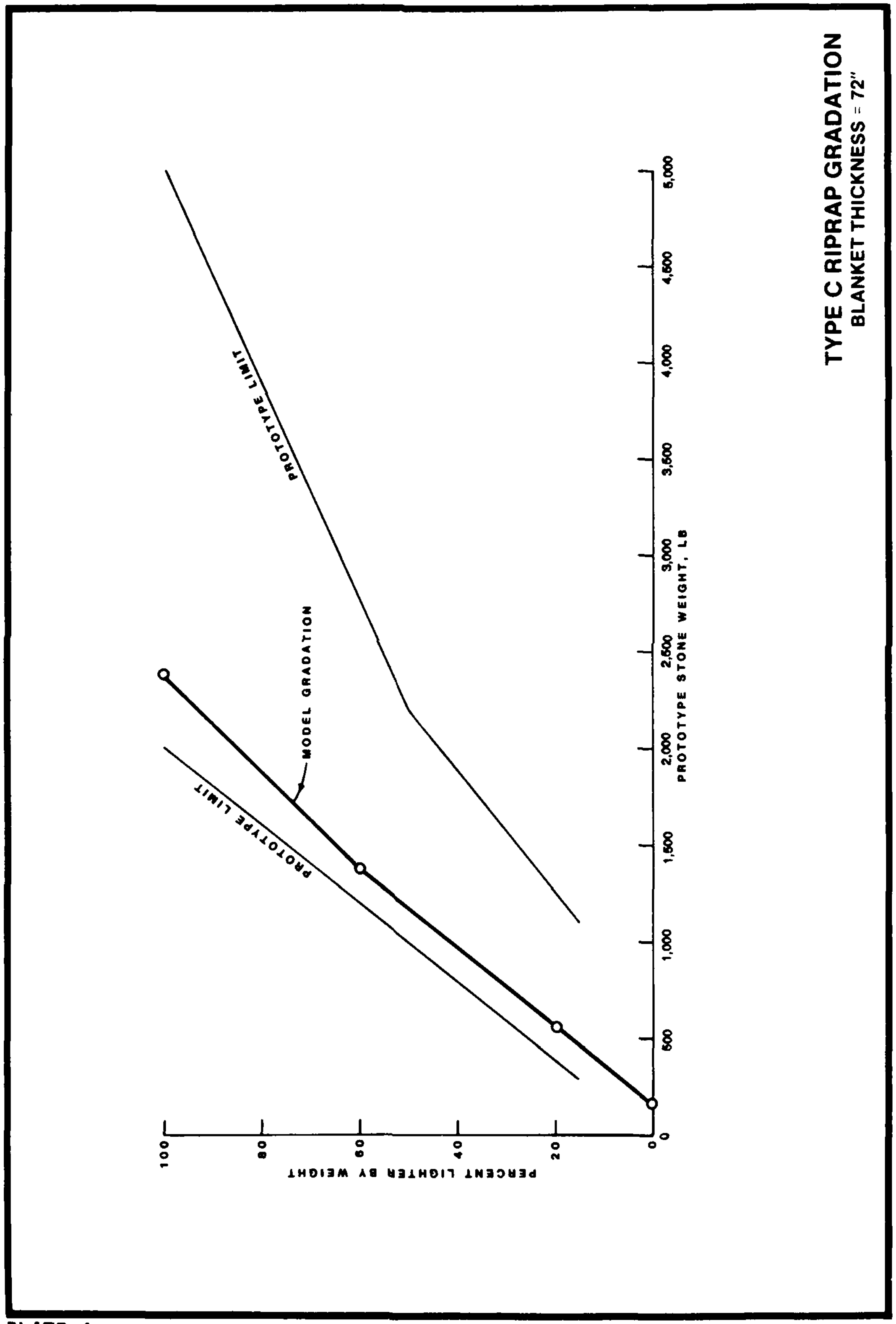
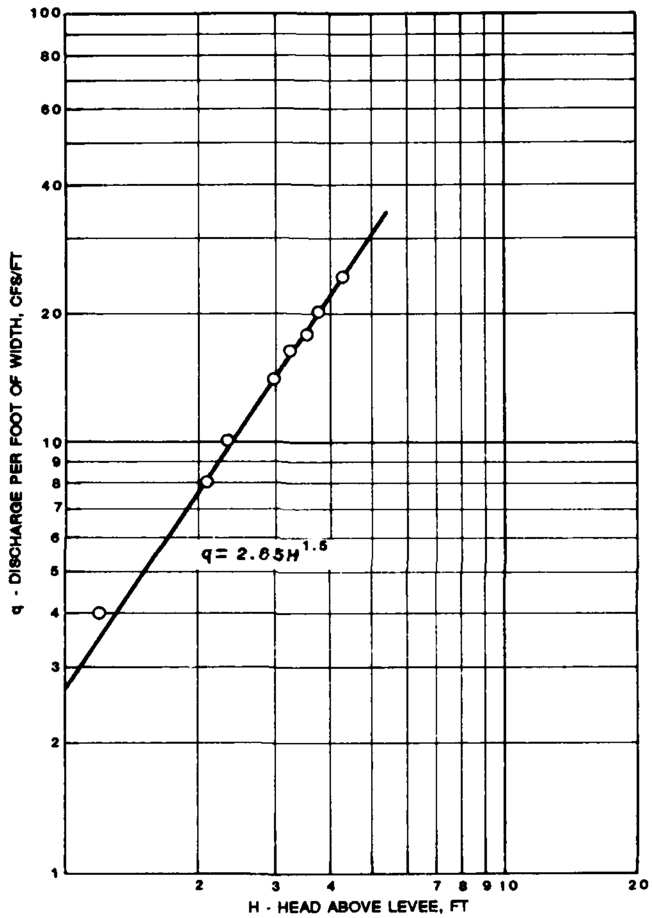
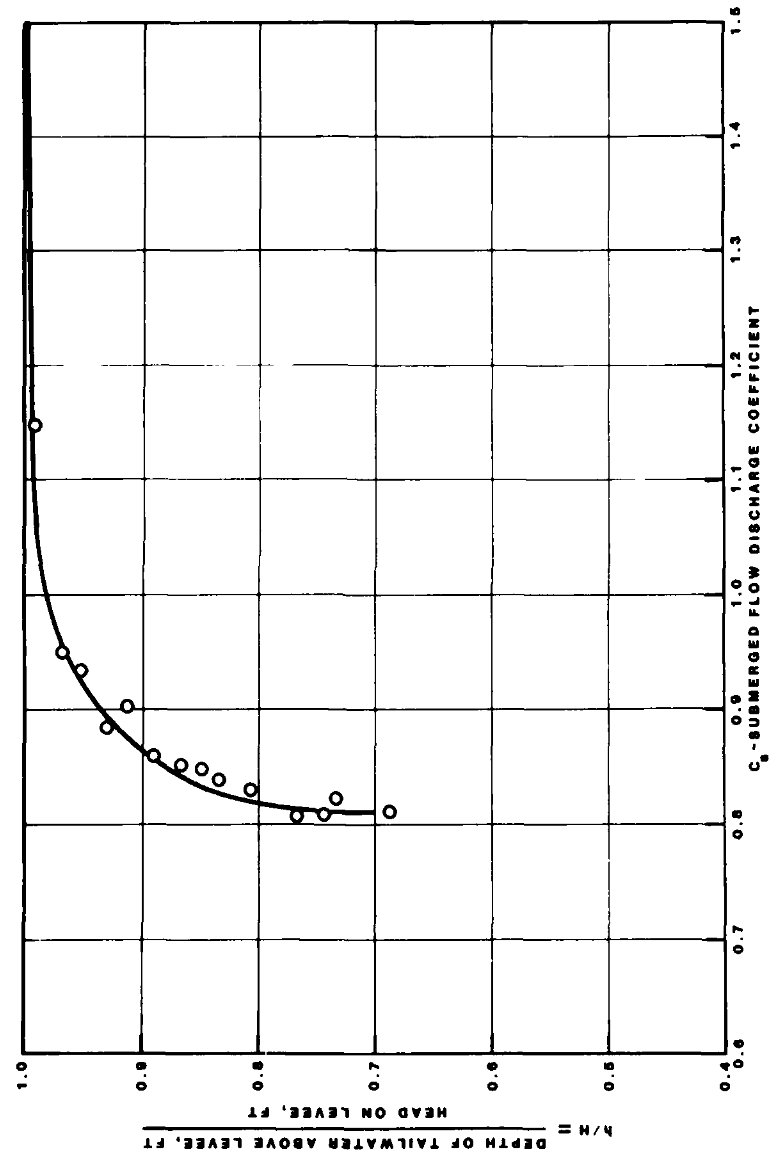

TYPE C RIPRAP GRADATION
BLANKET THICKNESS = 72"
PROTOTYPE LIMIT
MODEL GRADATION
PROTOTYPE LIMIT
PERCENT LIGHTER BY WEIGHT
100
80
60
40
20
0
PROTOTYPE STONE WEIGHT, LB
0
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
4,500
5,000

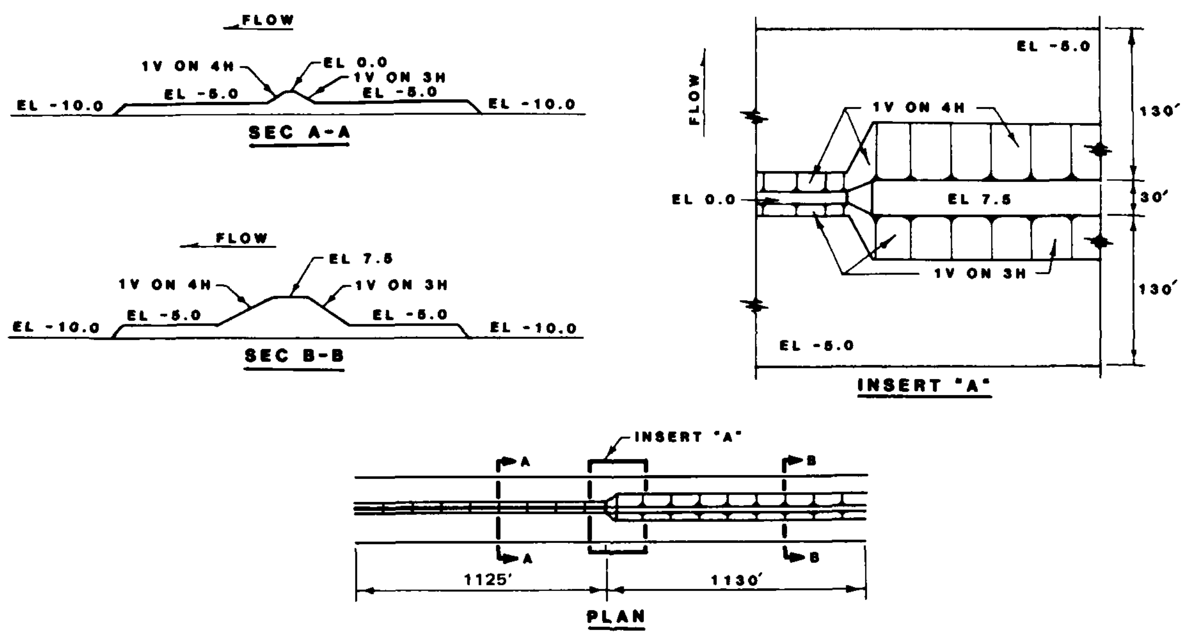
PLATE 4



PLAN A-2
DISCHARGE - HEAD RELATION FOR FREE FLOW
EL. 7.5 LEVEE

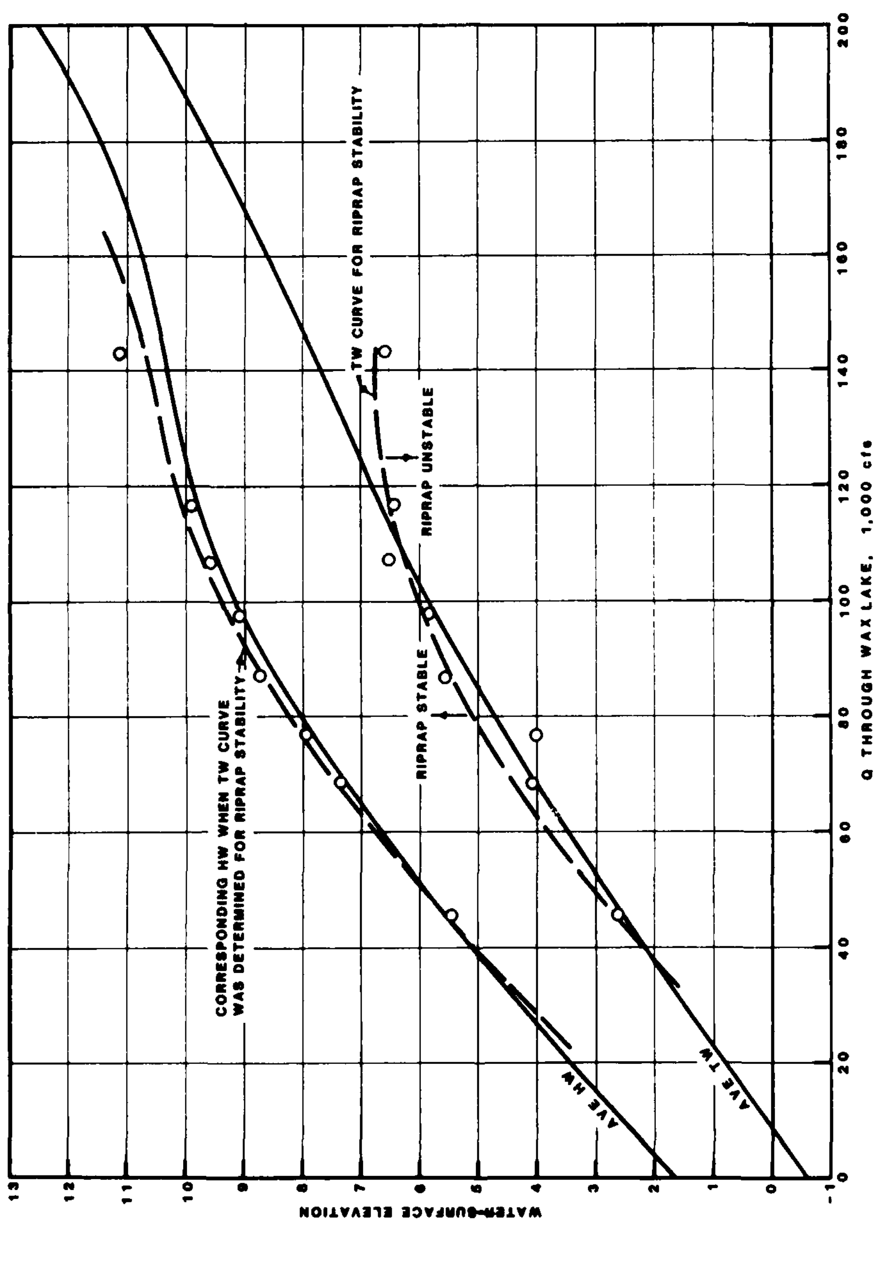


DISCHARGE COEFFICIENTS FOR SUBMERGED FLOW
EL. 7.5 LEVEE

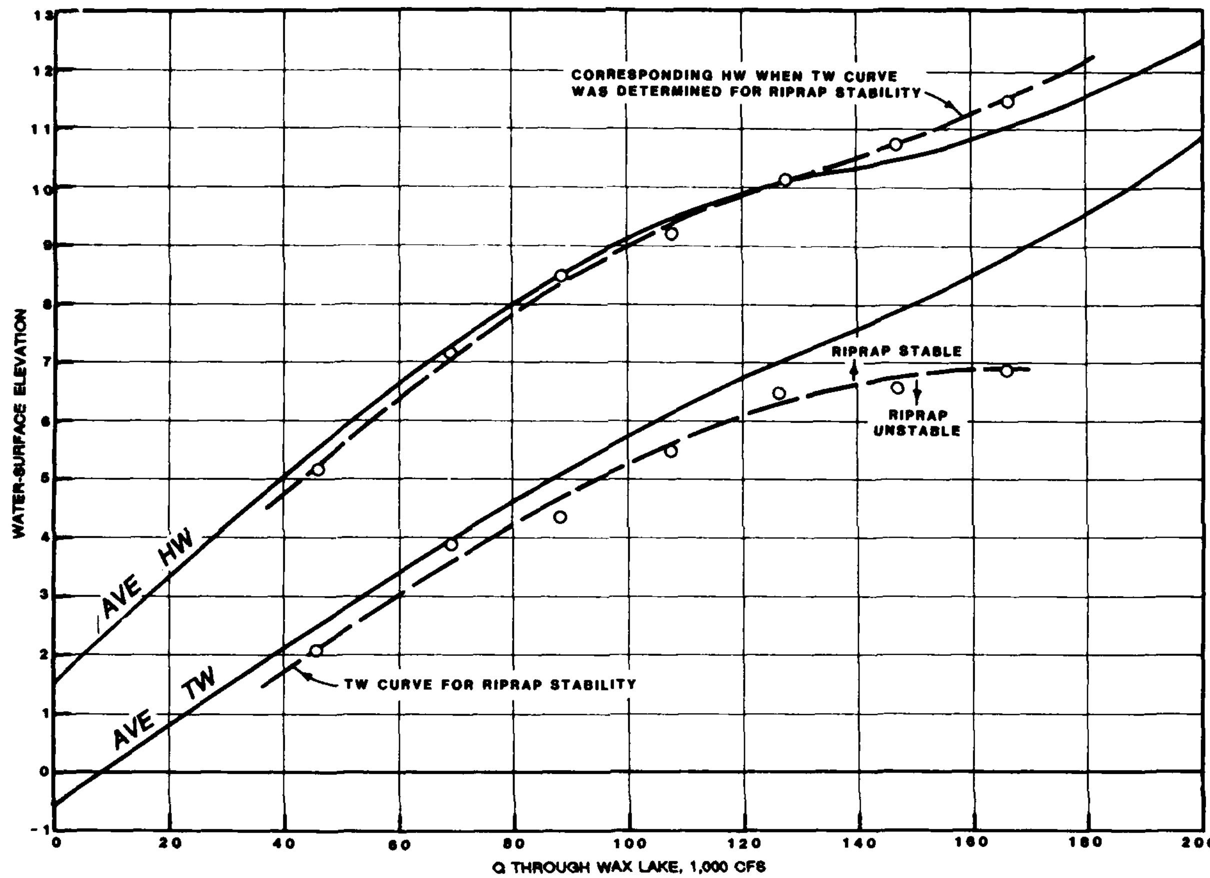


FLOW
1V ON 4H
EL 0.0
1V ON 3H
EL -5.0
EL -5.0
EL -10.0
EL -10.0
SEC A-A
FLOW
EL 7.5
1V ON 4H
1V ON 3H
EL -5.0
EL -5.0
EL -10.0
EL -10.0
SEC B-B
FLOW
EL -5.0
1V ON 4H
130′
EL 0.0
EL 7.5
30′
1V ON 3H
130′
EL -5.0
INSERT "A"
INSERT "A"
A
A
B
B
1125′
1130′
PLAN

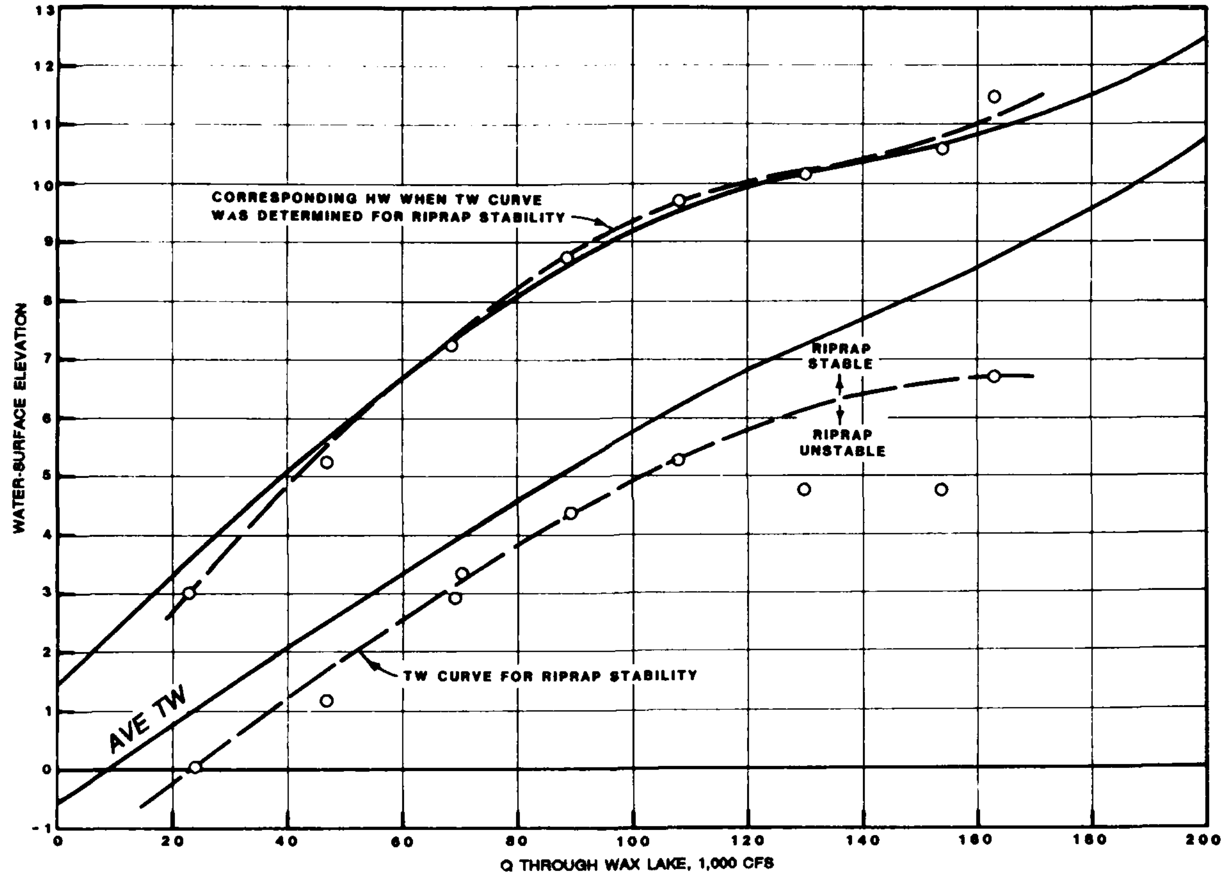
PLAN A-1



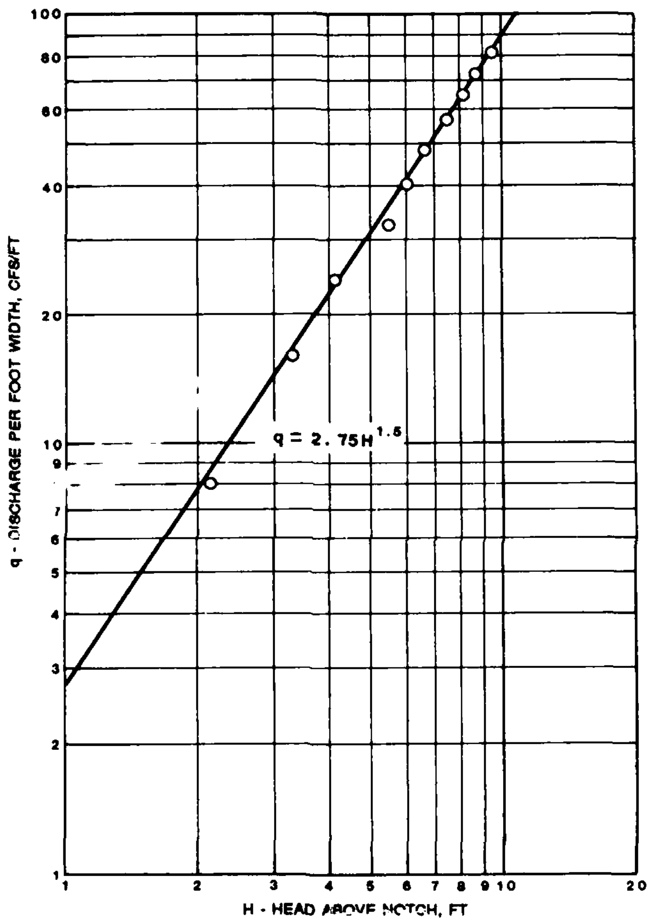
PLAN A-1
NOTCH RIPRAP STABILITY
TYPE B GRADATION
1 On 4 SLOPE ON DOWNSTREAM FACE



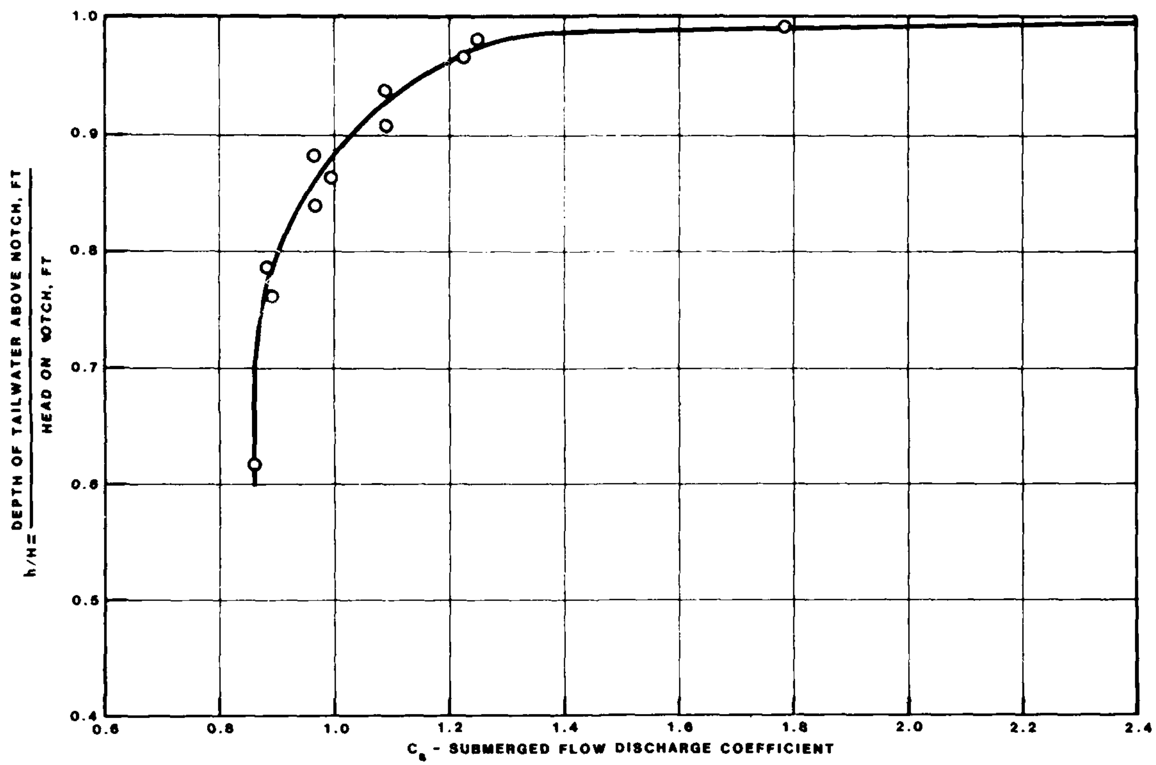
PLAN A-1
NOTCH RIPRAP STABILITY
TYPE C GRADATION
1 On 4 SLOPE ON DOWNSTREAM SLOPE



PLAN A-1
NOTCH RIPRAP STABILITY
TYPE C GRADATION
1 On 10 SLOPE ON DOWNSTREAM FACE

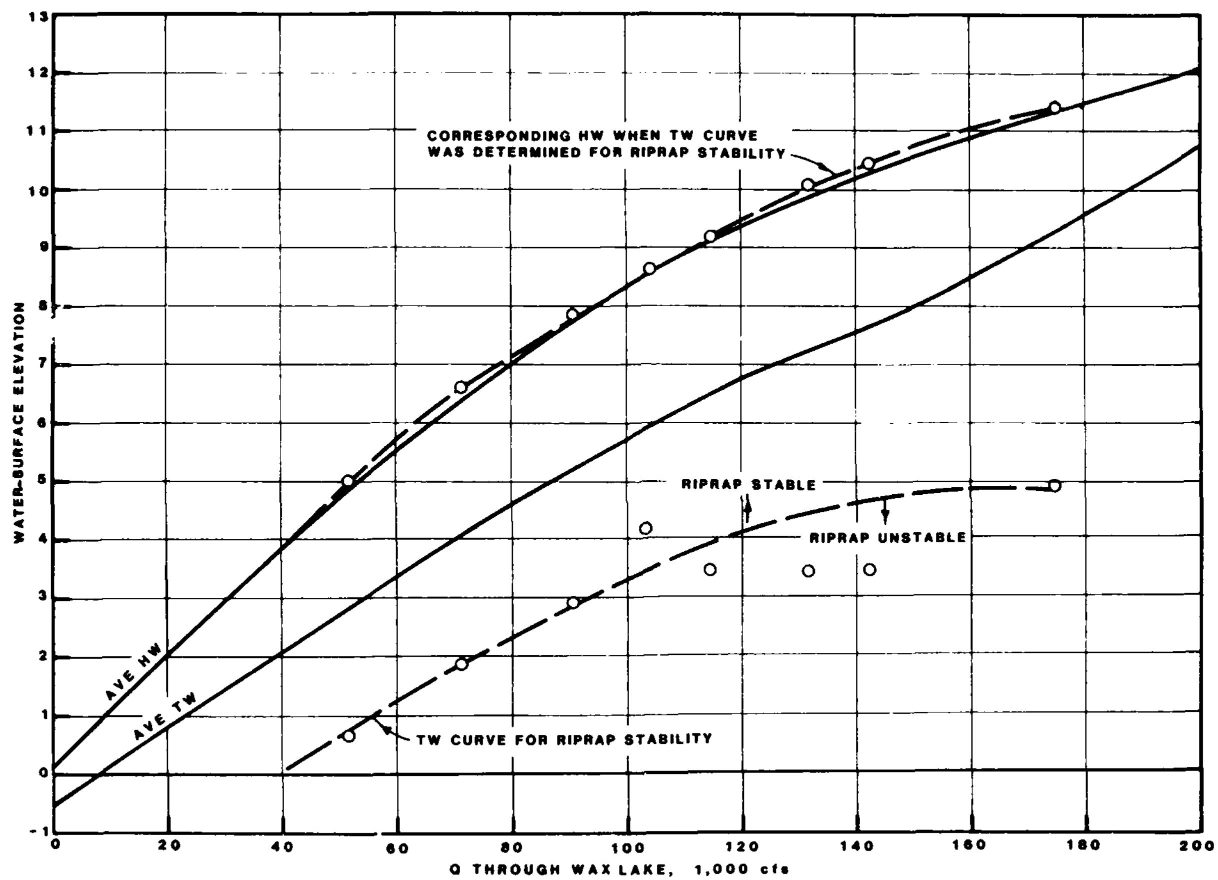


PLAN A-2
DISCHARGE - HEAD RELATION
FOR FREE FLOW
EL. -2.0 NOTCH

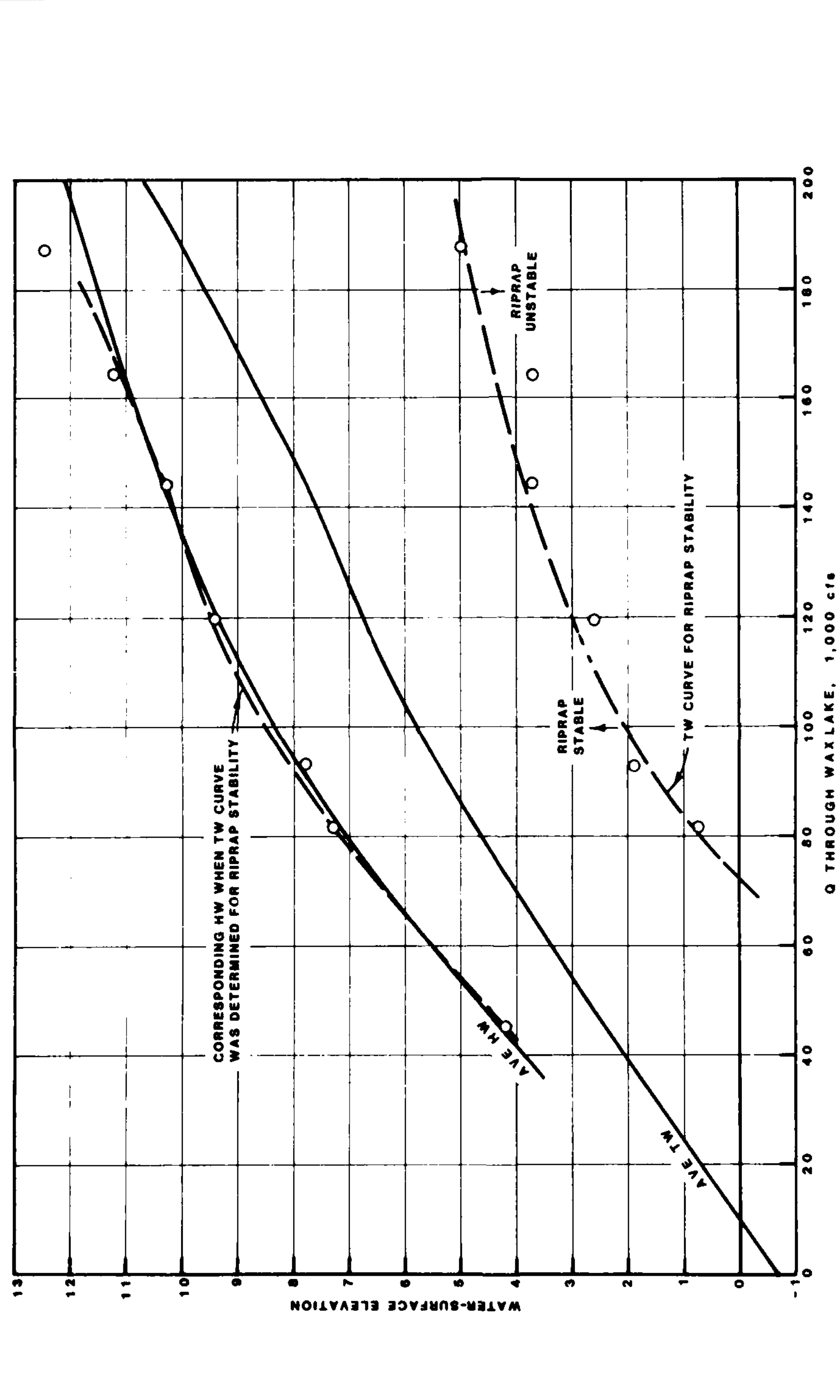


DISCHARGE COEFFICIENTS FOR SUBMERGED FLOW

EL. -2.0 NOTCH

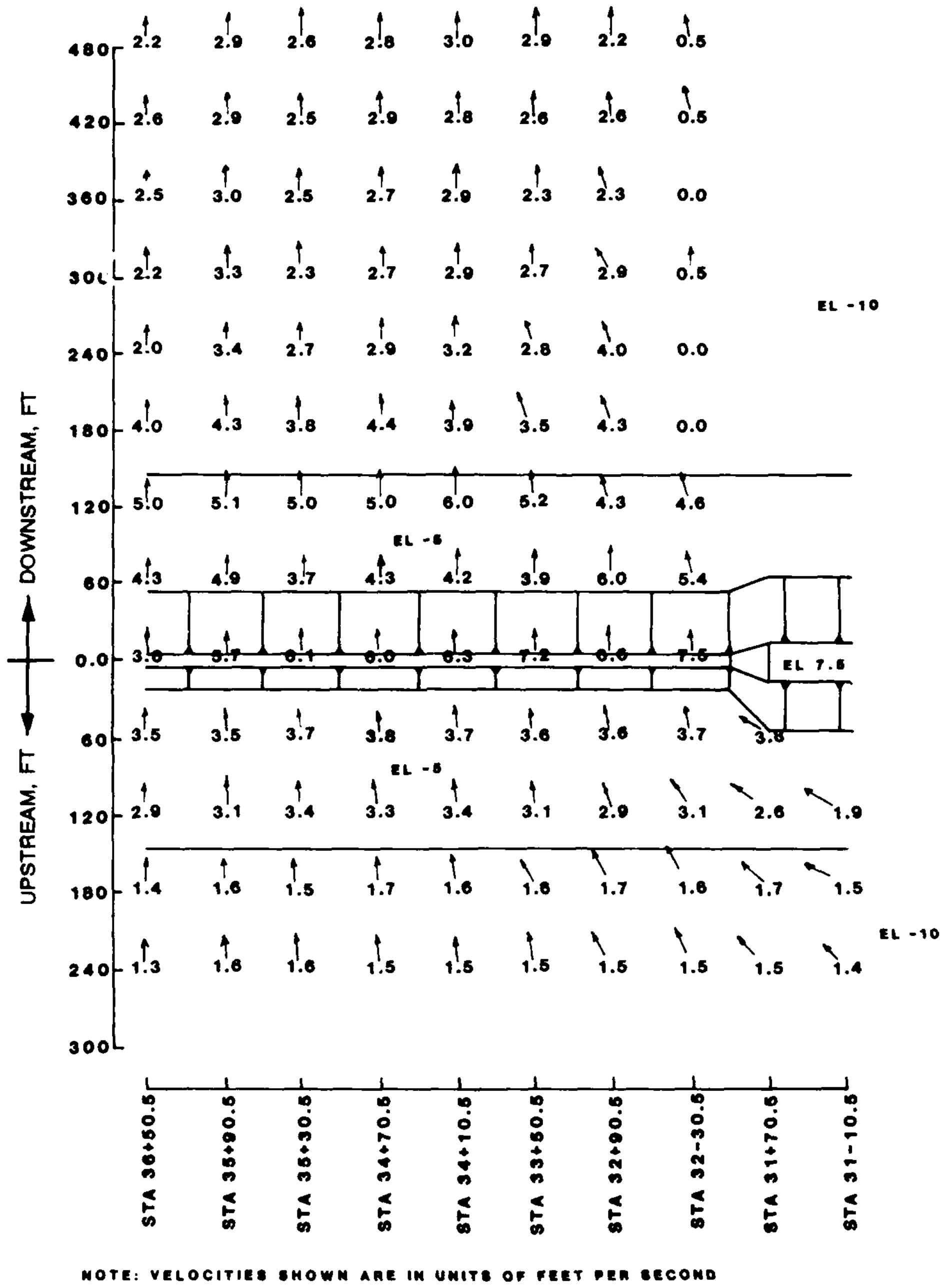


PLAN A-2
NOTCH RIPRAP STABILITY
TYPE C GRADATION
1 On 4 SLOPE ON DOWNSTREAM FACE



PLAN A-2
NOTCH RIPRAP STABILITY
TYPE C GRADATION
1 On 10 SLOPE ON DOWNSTREAM FACE

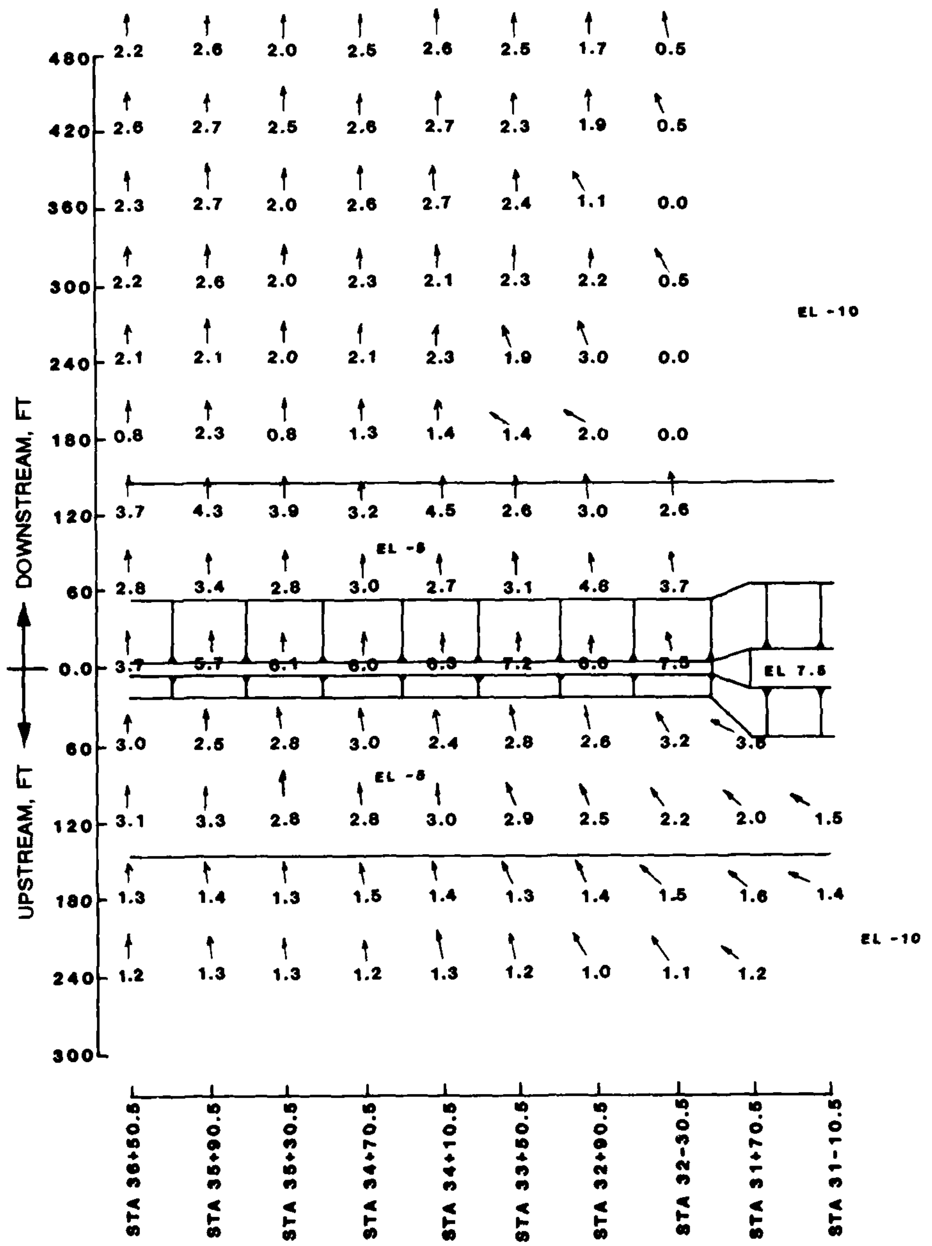
PLATE 14



PLAN A-2

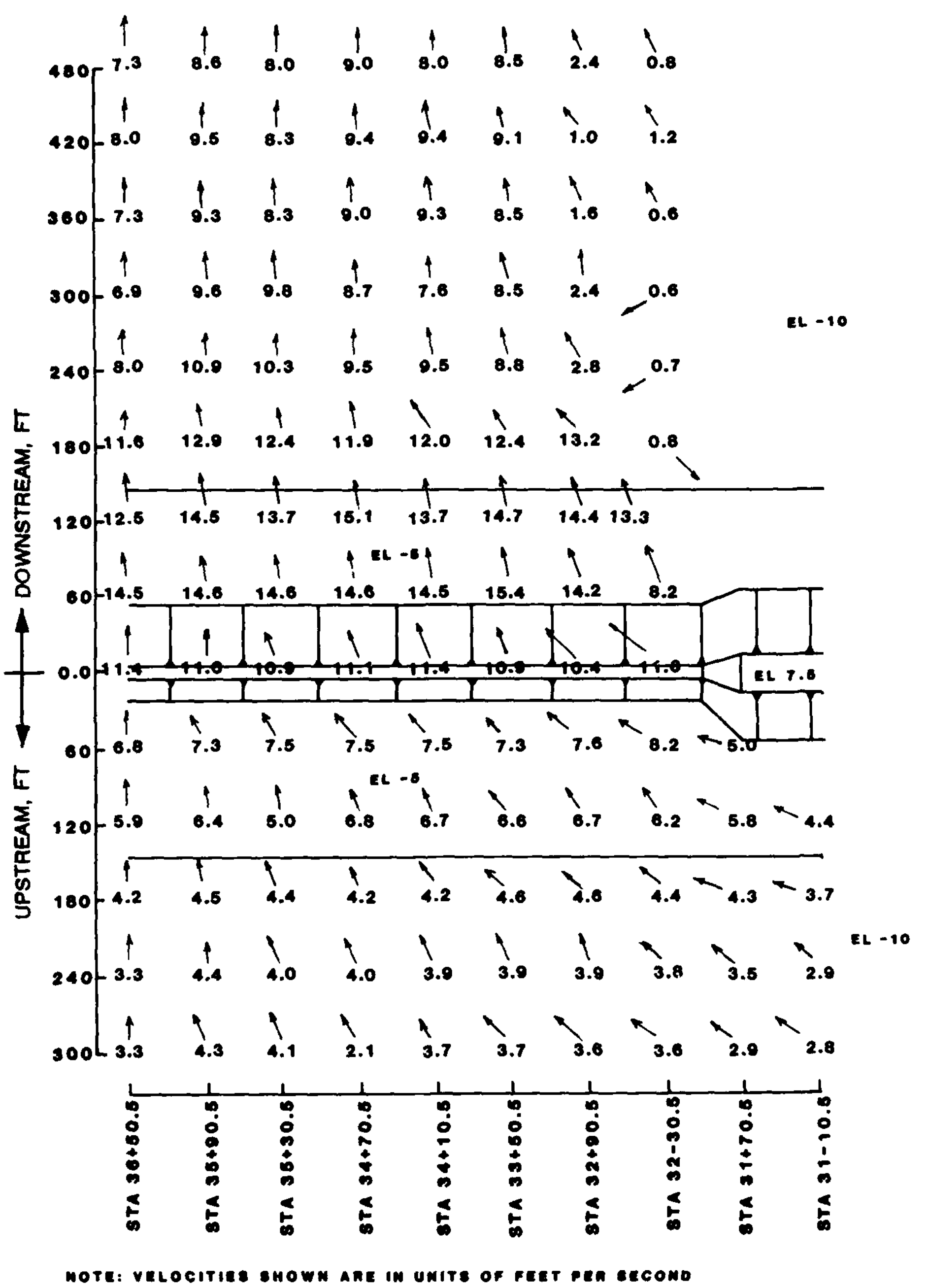
DISCHARGE THROUGH WAX LAKE 30,000 cfs

VELOCITIES OBTAINED 1 FT BELOW SURFACE

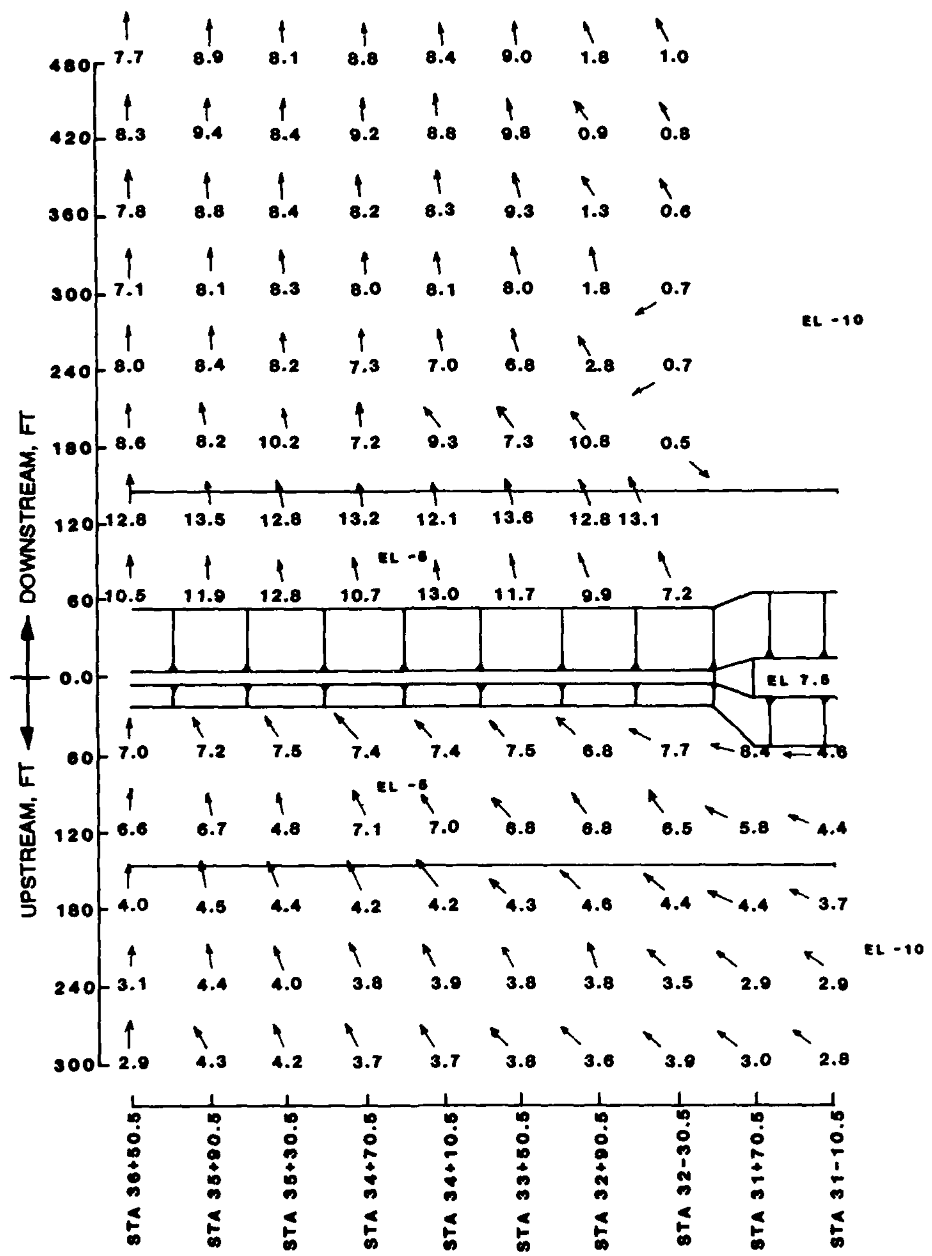


NOTE: VELOCITIES SHOWN ARE IN UNITS OF FEET PER SECOND

PLAN A-2
DISCHARGE THROUGH WAX LAKE 30,000 cfs
VELOCITIES 1 FT ABOVE BOTTOM



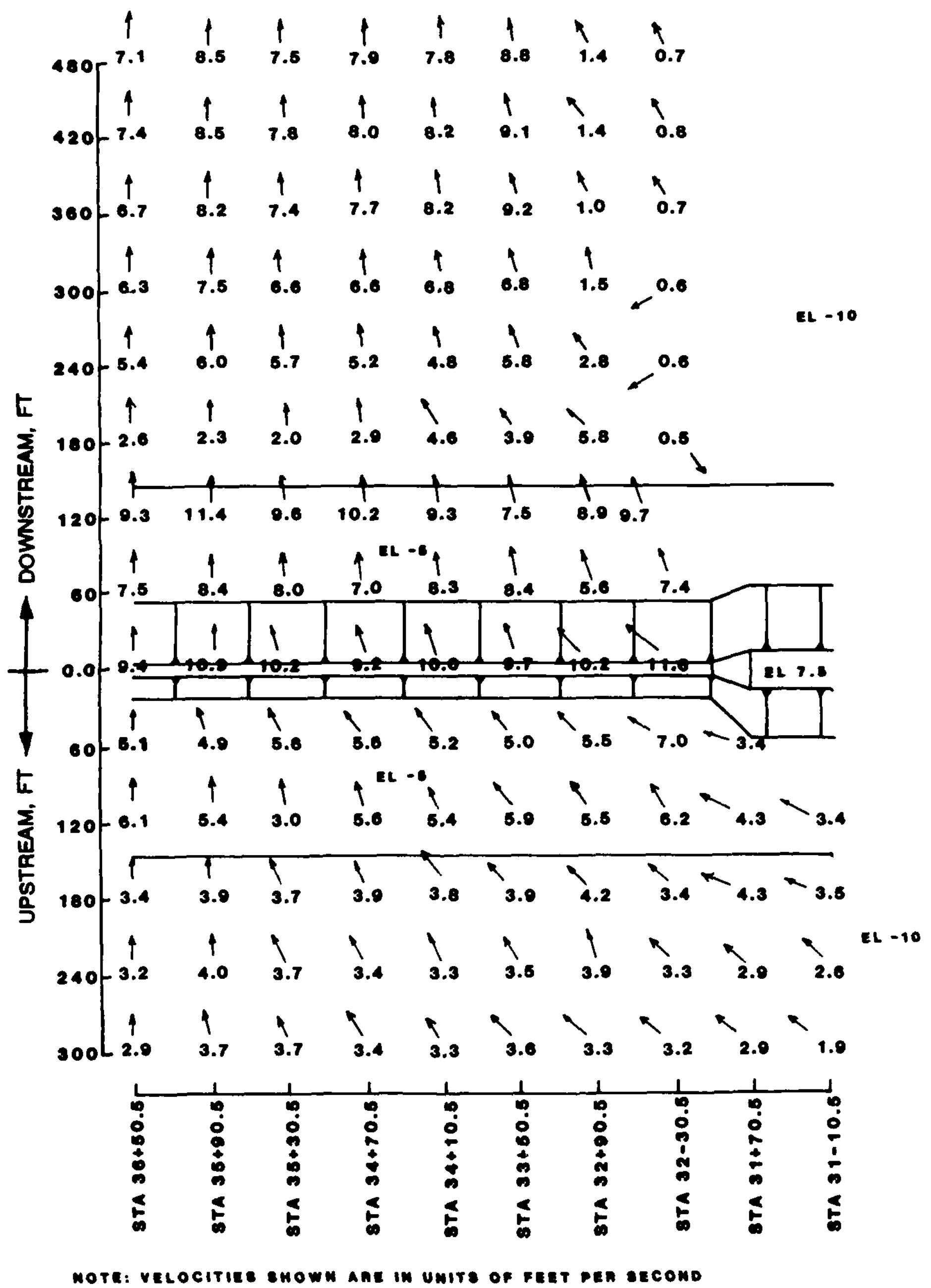
PLAN A-2
DISCHARGE THROUGH WAX LAKE 90,000 cfs
VELOCITIES OBTAINED 1 FT BELOW SURFACE



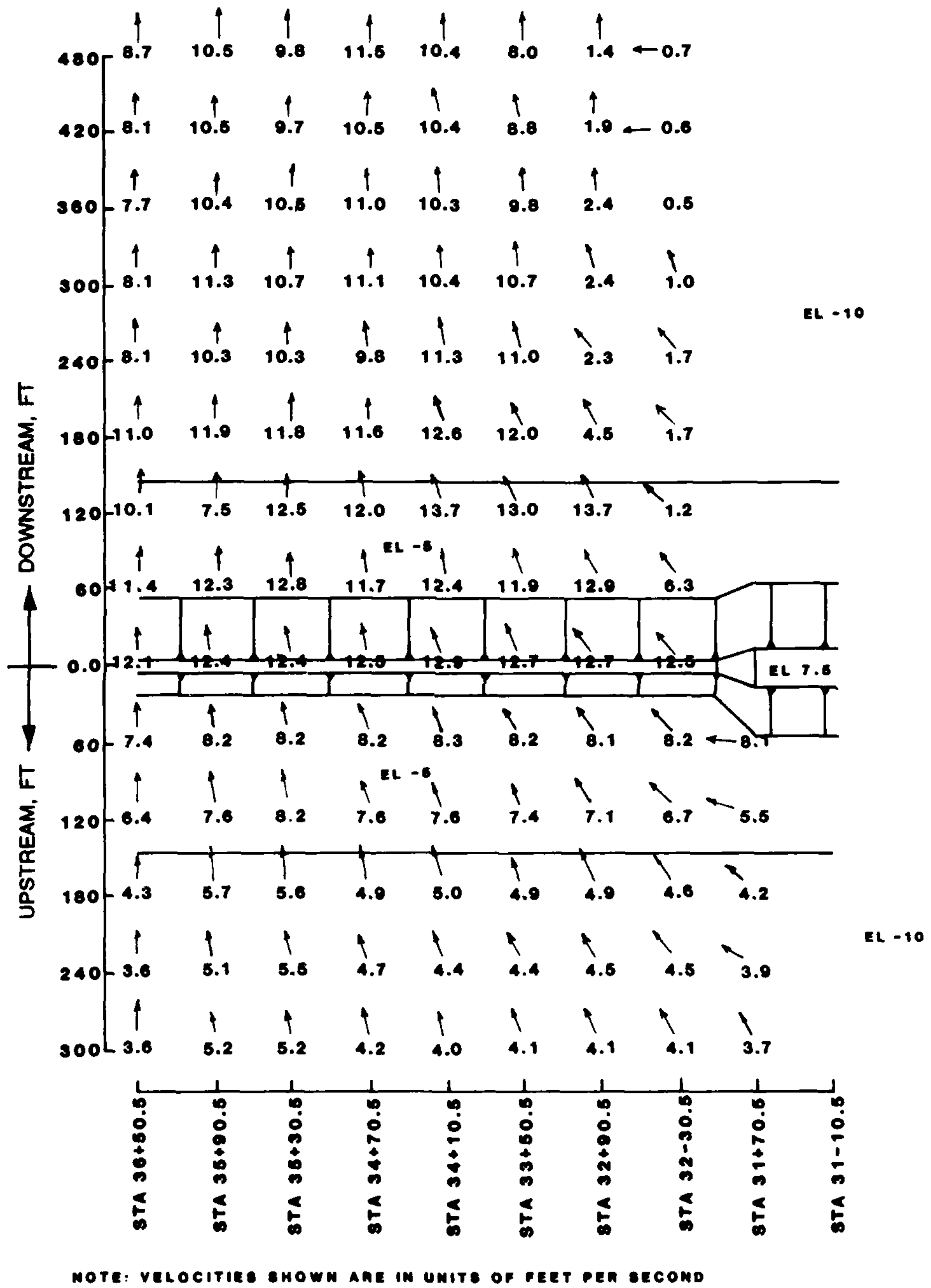
NOTE: VELOCITIES SHOWN ARE IN UNITS OF FEET PER SECOND

PLAN A-2

DISCHARGE THROUGH WAX LAKE 90,000 cfs
VELOCITIES OBTAINED AT MIDDEPTH



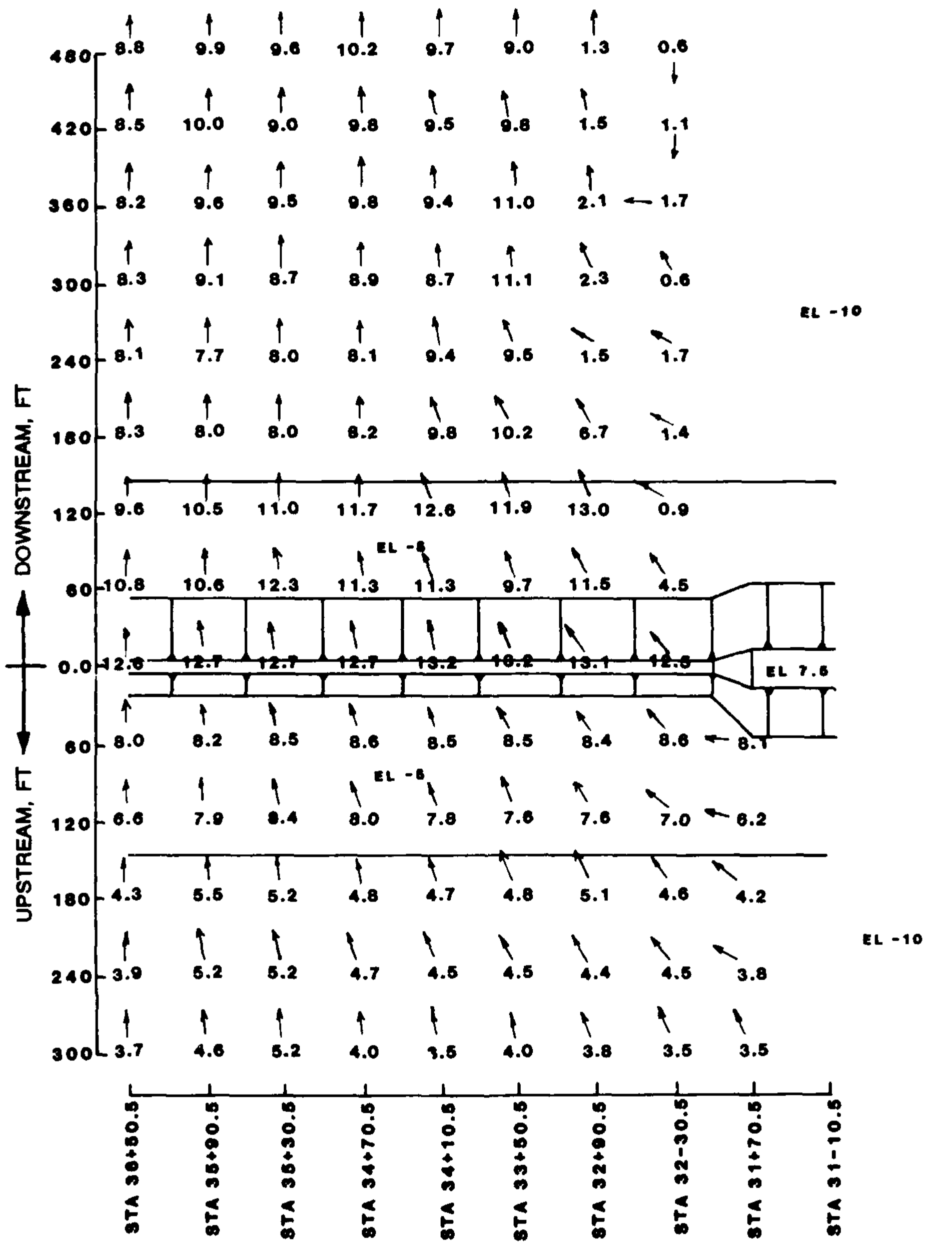
PLAN A-2
DISCHARGE THROUGH WAX LAKE 90,000 cfs
VELOCITIES 1 FT ABOVE BOTTOM



PLAN A-2

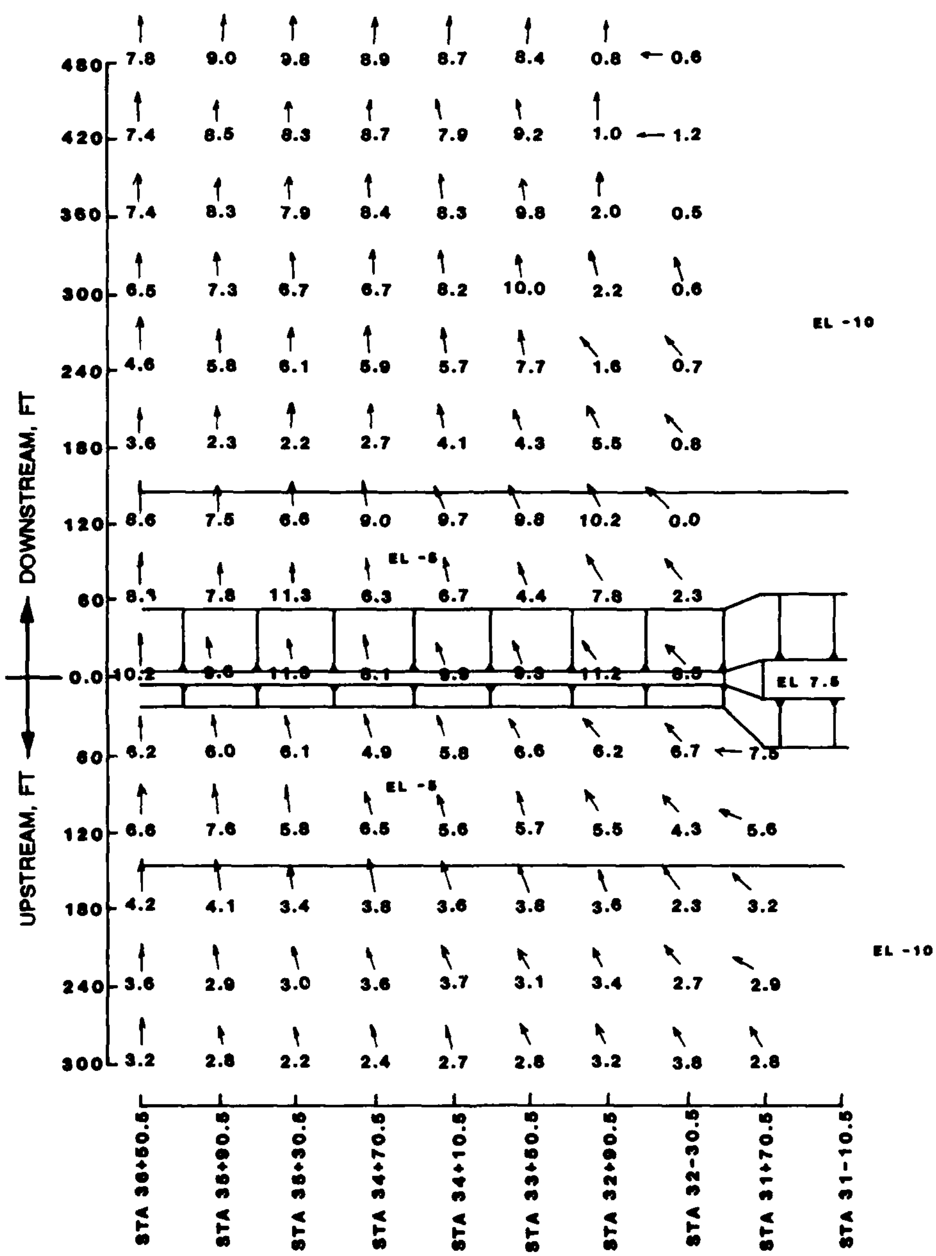
DISCHARGE THROUGH WAX LAKE 160,000 cfs

VELOCITIES OBTAINED 1 FT BELOW SURFACE



NOTE: VELOCITIES SHOWN ARE IN UNITS OF FEET PER SECOND

PLAN A-2
DISCHARGE THROUGH WAX LAKE 160,000 cfs
VELOCITIES OBTAINED AT MIDDEPTH

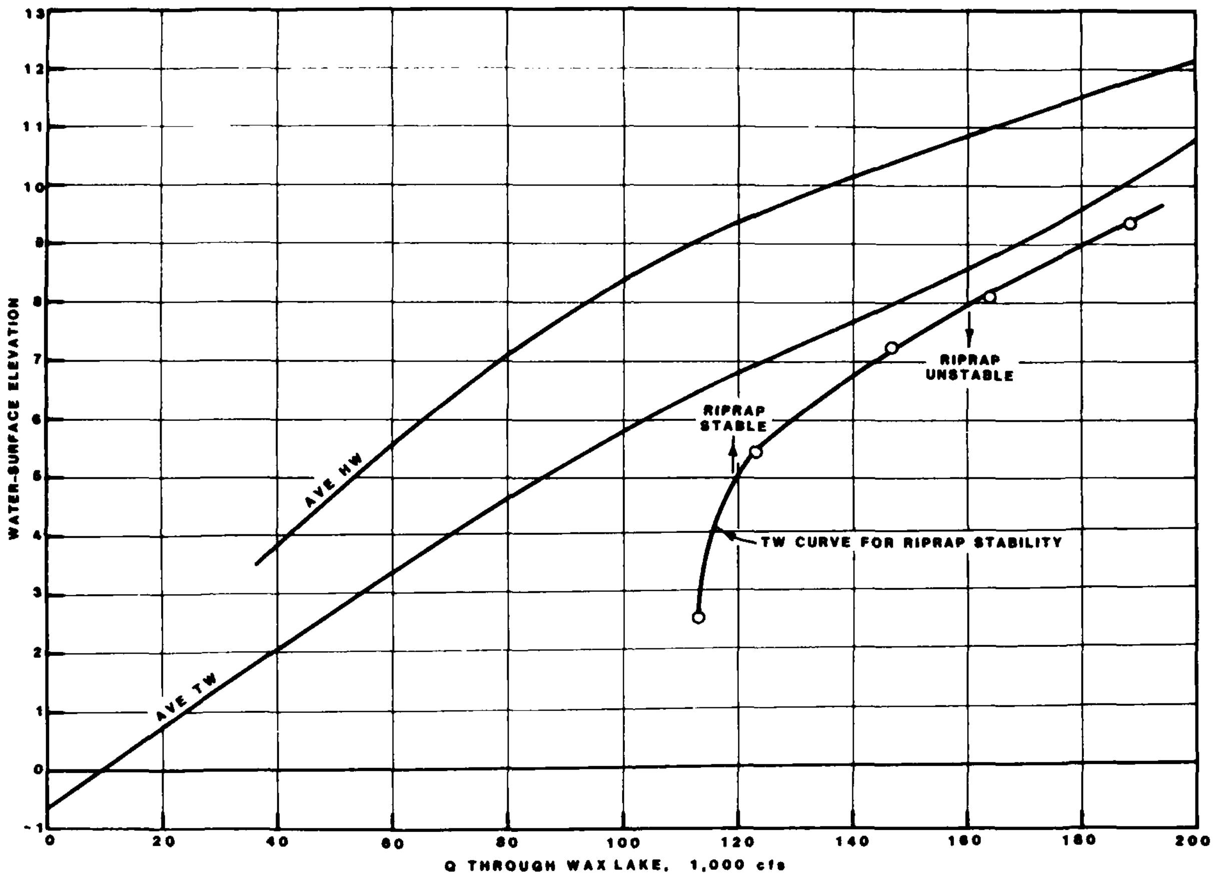


NOTE: VELOCITIES SHOWN ARE IN UNITS OF FEET PER SECOND

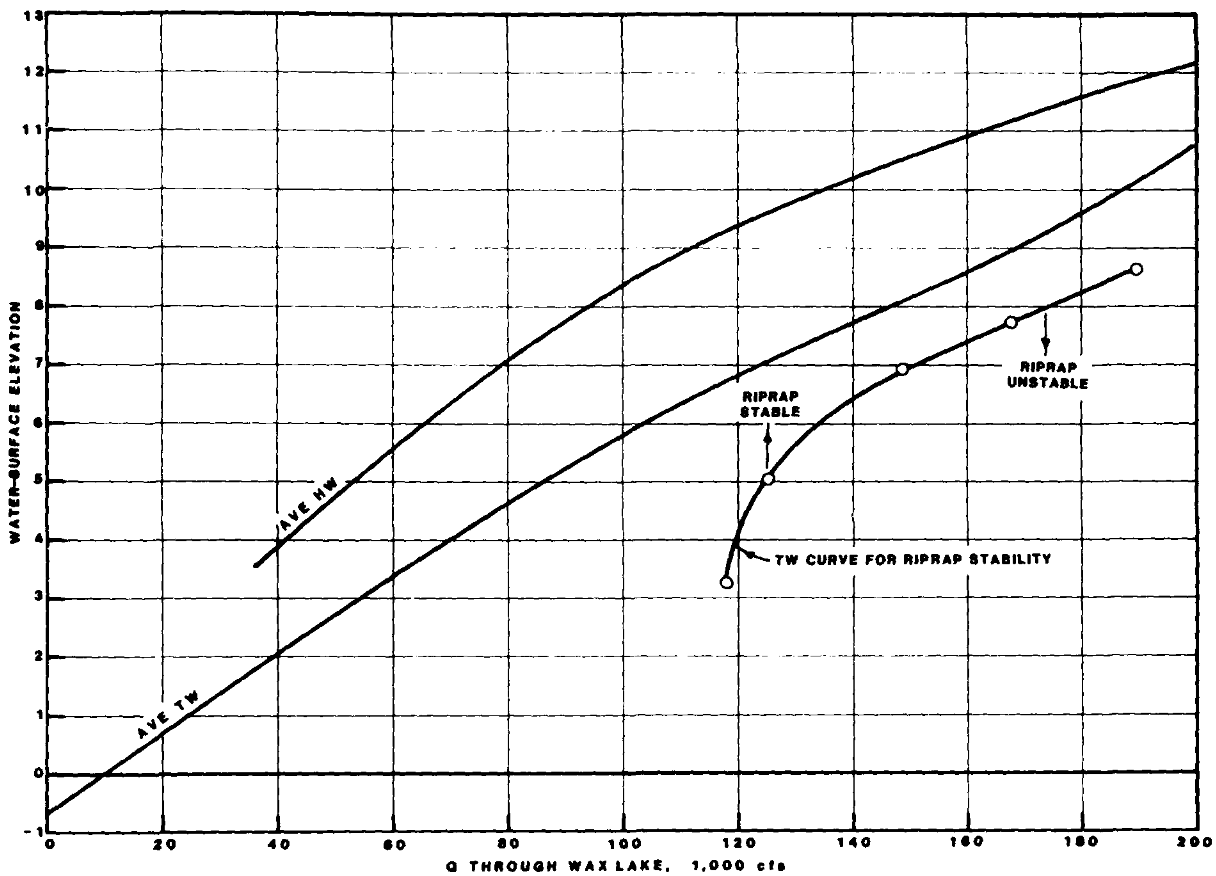
PLAN A-2

DISCHARGE THROUGH WAX LAKE 160,000 *cfs*

VELOCITIES 1 FT ABOVE BOTTOM



PLAN A-2
EL. 7.5 LEVEE RIPRAP STABILITY
TYPE A GRADATION



PLAN A-2
EL. 7.5 LEVEE RIPRAP STABILITY
TYPE B GRADATION